PROCEEDINGS
THE INSTITUTION OF
CIVIL ENGINEERS

PART II
OCTOBER 1954

AIRPORT ENGINEERING DIVISION MEETING

30 March, 1954

Sir George Fretwell, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Airport Paper No. 25

"The Construction of a New Runway at Amman, Jordan" *

by

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SYNOPSIS

The existing airfield at Amman was first used by the Turks and Germans during the 1914–18 war, and has been a Royal Air Force station since 1920. The airfield, which is small and unsuitable for heavy aircraft, occupies one of the few suitable sites to be found near the town of Amman.

The need for a new runway at Amman had been considered from time to time, but had been turned down on account of the general unsuitability of the terrain and the difficulties and high cost of construction.

In 1951 a site investigation was made and, although it was not possible to build an airfield to full flying specifications, the need for a new runway had become so urgent that Air Ministry gave approval for a runway to an acceptable standard to be constructed south of the existing airfield.

A description is given of the difficulties encountered in the construction of the runway arising from the nature of the site and the geographical position of Amman.

Some details are given of the site soil survey, of the materials used, and of the design of the flexible pavement.

The various operations involved in the construction of the runway and taxi-track are discussed.

Because of the large numbers of unemployed in Jordan, the contractor was required,

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under the terms of the contract, to restrict the use of mechanical plant to the minimum consistent with satisfactory workmanship. Whilst this requirement was met so far as possible, it was nevertheless, necessary to use mechanical plant for the larger operations.

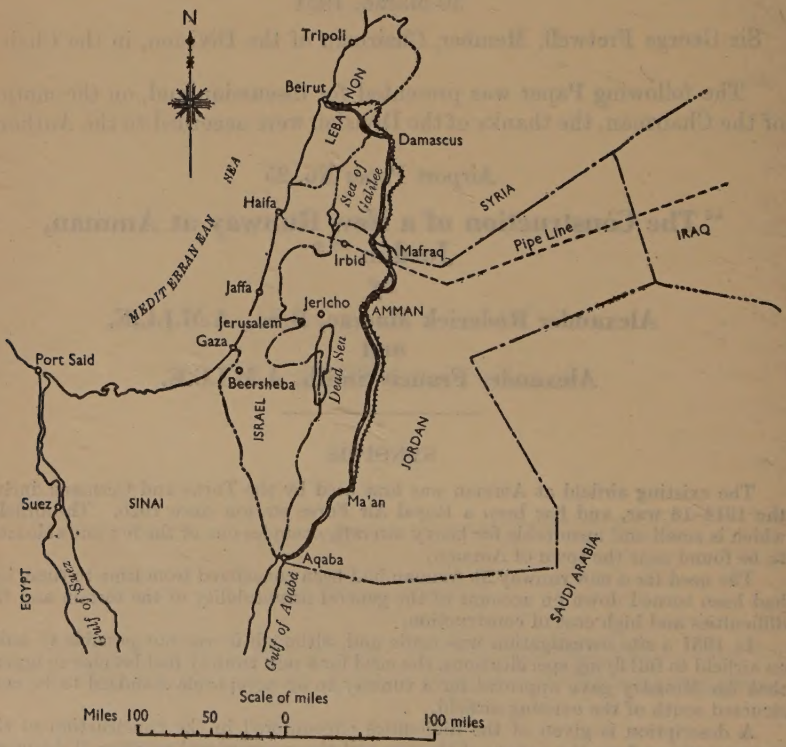
The largest single operation was the excavation of 1·4 million cubic yards of material, and reference is made to this in the Paper and to the plant used on the contract.

The construction programme is discussed, and an account is given of the deviation from the programme which actual conditions necessitated. Finally, brief details are given of the testing of materials on the site and elsewhere.

Introduction

The Paper describes the construction of a new runway at Amman, Jordan, and the problems encountered in its execution. It cannot be

Fig. 1



Map of Jordan

claimed that any of these problems were of special engineering interest or value, or even that they were of a kind unfamiliar to British engineers, but the location of the site and the fact that no similar work of anything like comparable extent had ever been carried out in that country previously,

did give rise to particular problems which are, fortunately, not normally encountered on similar contracts in Britain.

It was realized, when the runway was being planned in 1951, that there was no established contracting organization within the territory which could either assume complete responsibility for the work or give useful subcontracting services. Since no such organization already existed, there was no skilled labour force to draw on, and it was obvious from the outset that any contractor from abroad would have to bring a nucleus of skilled labour with him despite a Government keen to safeguard the interests of large numbers of local unemployed. He would also have to import the whole of his heavy plant and much of his material and, if sea journeys were involved, he would have the choice of only two entry ports, namely, Aqaba on the Red Sea and Beirut on the Mediterranean, each with its own peculiar difficulties. (See map, *Fig. 1.*) Aqaba, 225 miles from Amman, is the only port in Jordan itself. The Mediterranean ports of Palestine were given to Israel under the uneasy armistice of 1949, and there is now no communication between Israel and Jordan. At Aqaba only shallow draft vessels and lighters can be brought to the dock side and there are no facilities for off-loading more than 2 tons in any one lift. The rail head at Ma'an is 60 miles away and the desert road between Aqaba and Ma'an is impassable in the heavy winter rains. The alternative port of Beirut in the Lebanon is about 200 miles from Amman. The dock facilities at Beirut are adequate, but both road and rail traffic to Jordan must pass through Syria and face considerable difficulties with Customs. Low and narrow snow tunnels preclude the transport of heavy plant by rail, and the mountainous road journeys have their hazards in winter.

Such were the initial difficulties a contractor had to face, against which he could find no local assets in Jordan other than suitable quarries and an immense pool of unskilled labour.

In June 1952 the Air Ministry accepted the tender of the Motherwell Bridge Contracting and Trading Co., of London, who were associated with the Company of that name in Beirut ; the organization had commitments from Egypt to the Persian Gulf but no previous major contracting experience in Jordan.

History and Development

The new airfield at Amman is the latest stage in a long association between the Royal Air Force and the lands of Palestine and Transjordan. This association is now governed by the 1948 Treaty between Great Britain and Transjordan which was drawn up after the Emir Abdullah became the first king. The boundaries of the kingdom were extended in 1949 to include the portion of Palestine, excluding the Gaza strip, which remained on the Arab side of the Demarcation Line laid down by the Mixed

Armistice Commission. This was at the request of the inhabitants of the annexed territory, and, whilst the territorial addition was small, the population of the new Hashemite Kingdom of Jordan was greatly increased. Amman acquired new importance in two ways, as the seat of government of the enlarged kingdom, and as a commercial centre attracting much of the displaced business of Palestine. The town has grown very rapidly, the achievements of its building industry have been remarkable, and the population has increased from about 50,000 in 1949 to more than 200,000 (at the end of 1953). This growth has created many problems, especially in the fields of public health and road traffic, and there are many obstacles to real prosperity, notably the lack of natural resources and industry in the country, and the appalling problems of the destitute refugees from Palestine.

In spite of these difficulties, both the Central and Municipal Governments have shown a real determination to develop Amman as much as possible and, in the present development, their desire for a modern airport as near as possible to the town coincided with the needs of the Royal Air Force for an airfield with modern facilities in that important area.

The conditions affecting the choice of site will be discussed later. It is sufficient to remark at this stage that the new runway has been built on land adjoining the site of the present R.A.F. station, a station which was first made by the Germans operating for the Turks in the 1914–18 war. Obsolete bombs, believed to have belonged to the Turks, were actually dug up in the course of the work. During the Palestine Campaign of the 1914–18 war the airfield and the railway nearby were repeatedly bombed by the R.A.F. At the end of 1920 the R.A.F. took over the airfield and it is from that date that the development of the present permanent station originated. Two lightly constructed runways of less than 2,000 feet in length, made in recent years, were quite unsuitable for modern air traffic and the station had, in consequence, become of little value to the R.A.F.; all transport traffic had to be handled on the desert airfield at Mafraq, about 44 miles north-east of Amman, which is also used as a stopping place by B.O.A.C. for one of their regular services between London and Baghdad.

The existing Amman airfield was also in use by the Arab Legion Air Force and by civil traffic. Two local air transport companies were operating with de Havilland Rapides and Dakotas, the latter being the largest aircraft which could land and take off at Amman with safety.

All these factors emphasized the need for a modern airfield in the vicinity of Amman, the capital of Jordan.

The existing airfield at Amman is situated east of the town, on a small plateau bounded immediately to the west and north by a deep valley, whilst to the east and south the ground rises gradually to the surrounding hills which are only about a mile away.

The question of extending the airfield or constructing a new runway

had been considered on several occasions previously, but the restricted site of the existing airfield, and the uneven and difficult areas outside the airfield had the combined effect of condemning all proposed schemes as likely to prove too costly, if not entirely impossible.

By 1951, however, the Air Ministry were forced to the decision that, despite the general topographical unsuitability of the locality and the high cost of construction, a new runway to modern standards would have to be provided if at all possible in the vicinity of the present R.A.F. station at Amman to meet the requirements of the Royal Air Force. A new runway on, or adjacent to, the existing airfield would not only meet Air Ministry requirements but would, if made available for civil aircraft, also meet the Jordan Government's requirements as a first step towards a modern civil airport at Amman.

Accordingly, instructions were given for the area immediately to the south of the existing airfield, the only possible site in the immediate vicinity, to be surveyed with a view to the construction of a new runway capable of taking modern aircraft of all types.

In the immediate vicinity of the existing airfield, one east-to-west runway would suffice to meet all requirements, since experience had shown that practically all landings and take-offs are made on the east-to-west runway.

Site Investigations

The site investigation was carried out at Amman during the spring of 1951, closely following the recommendations of the Civil Engineering Code of Practice No. 1 (1951).

The main work of the site-investigation team consisted of topographical and soil surveys but, in addition, the following were investigated :—

(1) Water supply.
(2) Drainage.
(3) Access to site.
(4) Climate (effect on runway alignment).
(5) Sources of materials.
(6) Availability of unskilled labour.
(7) Availability of plant.

The survey team had a well-equipped soils laboratory, and local employees were soon trained to carry out routine soils tests.

As would be expected, the study of soil mechanics is almost unknown in Jordan, and it was necessary to train all the laboratory staff from scratch. It was not even possible to obtain staff with previous laboratory experience of any kind. The laboratory staff were trained at another site in Jordan where another airfield site was being investigated. The men trained for the laboratory work were building foremen, and the main

conditions of service were that they could weigh small quantities of materials accurately, and that they had a good knowledge of arithmetic. One man was trained for each test by the civil engineer in charge, the assistants carrying out the work automatically without knowing the purpose of the test. They quickly became very accurate in their work and were able to do a great number of tests per day; for example, one foreman and three labourers were able to complete eighteen C.B.R./density/moisture-content tests per 8-hour day, and one foreman and one labourer were able to complete seven liquid and plastic limits, or nine sieving tests from $\frac{1}{4}$ inch to B.S. sieve 200, or twenty field densities. The interested and better type of man soon learned extra soil tests, and it was possible to carry out the soil survey at Amman with four foremen.

Soils exploration was carried out entirely by means of trial pits. Normally a great deal of site exploration is done by means of small-diameter bore holes but because there was only one experienced engineer on the site to supervise both the field and laboratory work, it was decided to carry out the exploration with trial pits, thus enabling the engineer to see the soil section in its natural condition. Had bore holes been used, it would have been necessary for the engineer to supervise each hole as it was bored.

Pits were dug at 600-foot centres each way over the site, the depths being designed to investigate the soils to a depth of at least 10 feet below the finished level of the pavement. This required pits up to 24 feet in depth in deep-cut areas. It is the local custom to dig pits deeper than this without planking and strutting, and none was used at Amman. The pits were circular, because the local people are used to digging circular cesspits and cisterns.

Samples for the modified A.A.S.H.O. tests for the modified dry density and moulded C.B.R. tests were taken at the estimated formation level of the pavement in cut areas, and from the top 2 feet of soil in the case of fill areas. Field density tests were made at the estimated formation level and at 2 feet and 4 feet below this level. The soil from this test was also used for sieve analysis and the Atterberg limits. Field moisture tests were made every 2 feet below ground level. The core cutter method was used for the field density tests wherever possible, and the sand bottle method elsewhere. The results of the trial pit tests were recorded and a typical example is shown in Table 1.

The chief soil on the site was a highly plastic weathered calcareous clay and, although the colour of the soil varied from a dark reddish brown to almost white, the soil characteristics were similar. The weathered structure made the soil very dusty in the dry season and exceedingly sticky and unstable in the wet season. The average results of the standard soil tests were as follows :—

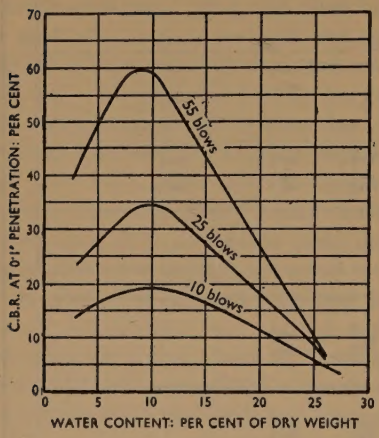
Field density	95 lb. per cubic foot	(average of 23 tests)
Maximum dry density	104 lb. per cubic foot	(„ „ 13 „)
Optimum moisture content	18·5 per cent	(„ „ 14 „)

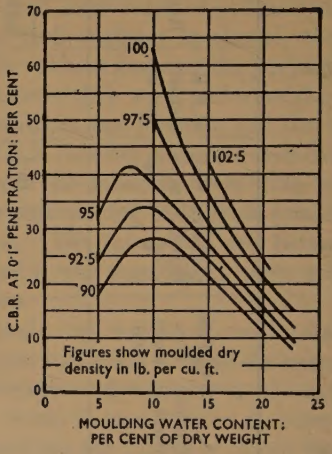
Table 1.—trial pit record

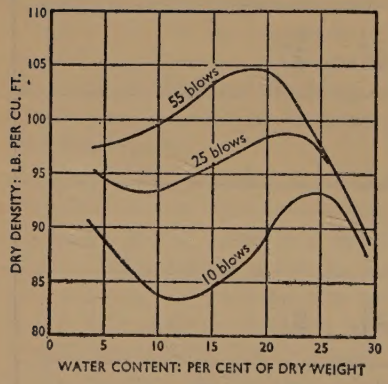
(April 1951)

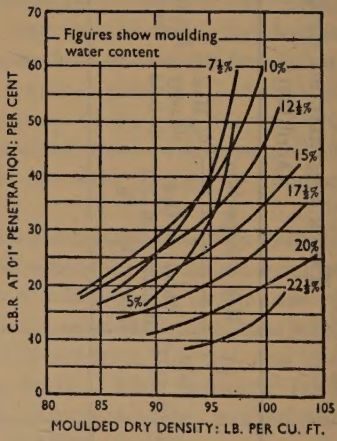
Depth below ground level (feet)	Soil profile	Soil class	Field density	Mod. compaction: Max. density: lb./cu. ft	Mod. compaction: Opt. m./c.: %	% Max. density in field	Liquid limit	Plastic limit	% Pass. B.S. sieve 200	Soaked C.B.R. %	Nat. m./c. %	C.B.R./density test no.	Remarks
0	Stiff light brown clay of high plasticity	C.H.									15·1		——Estimated formation level
2	Stiff reddish-brown clay of medium plasticity mottled with chalk	C.L.	102	108	15	94·6	47	25	89	4·0	18·2	11	
4			96			89·0	49	24	87		21·7		
6			97			90·0	52	27	93		24·1		
8	Light brown marly clay	C.H.									17·1		
10													
12	Bottom of pit												
14													

Figs 2









Details of tests :—

No. of layers	Blows per layer	Weight of hammer : lb.	Drop : inches
5	55	10	18
5	25	10	18
5	10	10	18

Notes :—

1. Specimens compacted in 6-inch-diameter mould. Tested as moulded.
2. These graphs are drawn as average results for three series of tests from a representative mixture of soil from top 2 feet of the following trial pits :—A16, A22, A28, A34, B10, B16, B22, B28, C10, C16, C22, C28, C34, P16, and P22.

GRAPHS SHOWING VARIATIONS IN C.B.R. VALUES WITH CHANGES IN DRY DENSITY AND MOISTURE CONTENT

Maximum dry density in the field . .	91 per cent	
Liquid limit	56 „ „	(average of 22 tests)
Plastic limit	29 „ „	(„ „ 24 „)
Percentage passing B.S. sieve 200 . .	90 „ „	(„ „ 24 „)
C.B.R. (after soaking for 4 days) . .	4·5	(„ „ 26 „)
Natural moisture content	19·2 per cent	(„ „ 35 „)
Casagrande group symbol	CH	
U.S.A. public roads administration classification,	A.7	

A series of C.B.R. and density tests were made to show the effect of moisture content on the C.B.R. value and density of the soil; the results of these tests are shown graphically in *Figs 2* by the method developed by the Waterways Experimental Station, Vicksburg, U.S.A.

The clay soil at Amman is not ideal filling material but economical construction demanded its use, since the site investigation did not reveal any material for use as hard filling other than quarried stone. It was, therefore, recommended that the fill under the pavement should be constructed with alternate layers of soil and broken stone, the layers being compacted with smooth-wheeled rollers on the stone, which would thus be forced into the clay. It was appreciated that by adopting this method of construction, the earthworks would have to be carried out during the dry season because the nature of the soil made it impossible to work plant in wet weather. In the dry season there is no water to spare at Amman to assist with compaction, so that there was also the risk of there being too little natural water in the clay for successful compaction in the summer.

To estimate the effect of hot dry weather on the compaction of the clay the soil was specially treated during the tests. The soil was broken into small pieces and spread on the concrete pavement in the sun for about 5 hours in a layer one inch thick. This reduced the moisture content to approximately 5 per cent. The modified compaction-test dry density for this moisture content averaged 95 lb. per cubic foot. To obtain this density, the same amount of work had to be done to the soil as for the maximum dry density at optimum moisture content.

It was ascertained that no plant was available in Jordan and very little in the Middle East. It was, therefore, advised that all the plant required for the construction would probably have to be imported from the United Kingdom.

At the time of the investigation there was a boom in residential building in Jordan resulting from the transfer of assets from Palestine. This boom, as it happened, died out by the middle of 1952 but at the time of the investigation it did seem that, whilst there would be an abundance of unskilled labour, there would be a shortage of local tradesmen for the runway throughout the whole of the contract.

As a result of the survey it was concluded that whilst a runway to ideal standards could not possibly be constructed in that area, it would nevertheless be possible to construct a runway to acceptable dimensions and standards.

On the strength of these conclusions the decision was made by the Air

Ministry to go ahead with the design and construction of a suitable runway with the necessary taxitracks.

Climate

The Royal Air Force Station at Amman is about 2,500 feet above sea level. The average maximum summer shade temperature is about 90° F., though for short periods it may reach or even exceed 100° F. Wind and dust are often troublesome in the summer and impede all work. Prevailing winds are westerly. There is no rain between March and late October, and most of the year's rainfall is normally concentrated in two or three winter months. In the winter of 1952–53, for example, the winter rains were concentrated in the months of February and March, during which there were several periods of a week or more when the site was waterlogged and work was at a standstill. Winter temperatures can fall to freezing point or slightly below, bringing hail, sleet, and bitter winds. There are times in the winter when the airfield is enveloped in cloud.

Description of Site

The site selected for the new runway and taxitrack is a narrow east–west strip immediately to the south of the existing Amman Airfield. South of the new flight strip and about a mile away is a range of hills in the form of a limestone ridge running parallel to, and about 500 feet higher than, the new runway. Two wadis or small valleys lay across the site draining the ridge and joining another wadi running parallel to the new flight strip on the north side of the site. There was a fall of over 100 feet between the western and eastern end of the site selected for the runway and of 30 feet in some places between the southern and northern edges of the new flight strip. The soils are mainly clay, becoming stony with depth, and overlying weathered limestone rock. The whole of the strip was very uneven, and within the strip length there were three hillocks and two valleys, which had to be cut and filled to form a uniform gradient of 1 in 80 along the whole length.

On the hillock at the west end of the site a village lay within the proposed runway area. There were approximately a thousand inhabitants in the village, the dwellings of which were mainly mud-brick with a few stone buildings. To make way for the new runway, the village was evacuated and demolished by the Jordan Government, and the inhabitants were compensated.

There was neither a sewage disposal scheme, nor a running water supply in the village, the custom having been to dig a water cistern and cesspit under each house. After the village was demolished, there remained 130 of these cisterns and pits up to 20 feet deep and 10 feet diameter. There were also three large caves formed under the desert crust which

were used to shelter cattle. Fortunately the village was in an excavation area and most of these pits were above finished formation level, but for the safety of the excavating plant they all had to be filled in before excavation could commence (*Fig. 3*, facing p. 408).

Also within the village area there was a Moslem cemetery containing 902 bodies. These had all to be exhumed and re-interred in three communal graves clear of Air Ministry land.

Design of Pavements

The main problem in lay-out design was to provide a runway of adequate length in such a restricted site, yet complying with the longitudinal and cross-fall gradients and having the necessary clearance within the approach funnels. In the lay-out finally approved (Fig. 4, Plate 1) the new runway length is 8,000 feet, the longitudinal gradient is the maximum permissible figure of 1 in 80, and the crossfall is 1 in 100. An over-run of 1,000 feet was possible at the eastern end of the new runway, but at the western end an over-run of only 300 feet between the end of the runway and the edge of the escarpment was possible. This was acceptable in view of the wind characteristics at Amman and of the gradient of the proposed runway.

Whilst the runway was planned and designed to meet R.A.F. requirements, the knowledge that it would also be used by civil airlines and that the airfield would in due course also be developed as a civil airport was taken into account at all stages of planning.

After consideration of all the factors, a flexible pavement design was adopted for the new runway. Even on soil of comparatively poor bearing value, such as the soil at Amman, a flexible pavement offers many advantages over a rigid pavement if only on account of the very high price of cement in Jordan. Since the runway had also to be designed for jet aircraft, it was decided, however, to provide concrete pavements in the areas liable to be subjected to the maximum jet blast.

The flexible pavement was designed on the Californian Bearing Ratio system and the rigid pavement design was based on the Air Ministry development of Westergaard's theory.

The bearing value of the soil is high at low moisture content but the field moisture content indicated that, in spite of the arid climate of Amman, high moisture content and low bearing value must be allowed for. It was, therefore, recommended that the flexible pavement should be designed on the soaked C.B.R. value and on this basis total thicknesses of 25 inches for the taxiways and runway ends and 22½ inches for the central portion of the runway were adopted. The surfacing of the flexible pavement was specified as 1-inch dense bituminous wearing coat on a 2½-inch bituminous macadam base coat.

Normally, a flexible pavement of the strength required at Amman

would require a sound well-compacted base course of which only the top 6 inches need be of well graded crushed aggregate, having a high C.B.R. value. Since there were no natural materials of suitable strength and grading in sufficient quantities within easy reach of the site, it was found necessary to use graded crushed stone for the full depth of the pavement underneath the bituminous macadam surfacing.

The thickness of the concrete areas was designed by Westergaard's original formula, as modified by Air Ministry. A flexural strength of 480 lb. per square inch for concrete and a coefficient of subgrade reaction k for the site of 100 lb. per square inch per inch were assumed. The thickness of the concrete slab required was more than 12 inches and it was, therefore, decided to construct the rigid pavement in two layers with the bottom layer 9 inches thick and the top layer 8 inches thick; slabs were to be separated with a layer of bitumen emulsion and the joints on the top slab were to be laid to break joint with those of the lower slab.

Sources of Supply

The chief materials required for construction were 7,500 tons of cement, 1,500 tons of bitumen, 155,000 cubic yards of crushed aggregate, 56,000 cubic yards of hand-broken stone, and 16,000 linear feet of large-diameter storm-water drainage pipes. Five hundred tons of cement were obtained from Syria, 1,500 tons from Jugoslavia, and the remainder from the Lebanon. All the bitumen was imported from Holland. The aggregates were crushed on or near the site from crystalline limestone obtained from nine quarries which were all located within 2 miles of the boundary of the site. Tests of the aggregate obtained showed a consistent crushing value of 28 per cent. It had been hoped to obtain the storm-water drainage pipes in Jordan but the pipes manufactured locally were well below specification requirements. The main storm-water drainage pipes were of spun concrete from the Lebanon. A great number of 4-inch- and 6-inch-diameter drainage pipes were cast in situ with "Ductube."

Plant and Labour Difficulties

Since in Jordan there were many people unemployed, the Air Ministry requested the tenderers to restrict the use of mechanical plant in the execution of the work to the minimum, consistent with satisfactory workmanship and so provide the fullest possible employment for local labour. The contractor met this request by hand breaking stone for hard filling, and by hand digging all drainage trenches.

As mentioned earlier, the nearest seaports available to Jordan are Aqaba and Beirut. The contractor chose Beirut for his off-loading port for all materials and plant coming from overseas to Amman. Almost all the mechanical plant and equipment required had to be obtained from

the United Kingdom. The route from Beirut to Amman passes through Syria, in which country there is a regulation that all non-perishable goods passing through the country must be conveyed by rail. In consequence, the Syrian railways are greatly overworked and considerable delays arose in the transport of the contractor's plant and equipment from Beirut to Amman. The special permission required for the materials and plant, too bulky for the railways, to be conveyed by road led to even further delays. A real difficulty was encountered in finding low loaders fit to transport scrapers, graders, and other large plant by road from Beirut to Amman. Early in the contract an attempt was made to bring a self-propelled scraper under its own power by road from Beirut to Amman, but this effort ended in disaster near Damascus, when the scraper slipped off the road crashing down a hill into a wood below. This machine was ultimately recovered and eventually reached Amman to be repaired and put to work on the contract nearly a year after leaving Beirut.

Although there were large numbers of unemployed in Amman, there were very few Diesel mechanics or plant operators among them. The Jordan Government limited the contractor to a maximum of 30 foreigners, inclusive of agent and staff, to be employed on the job. For the first few months of the contract, therefore, the contractor was obliged to spend a great deal of time and effort training local employees as plant operators and mechanics; the Jordanians soon became proficient and eventually they operated and largely maintained all the plant. The last type of plant to be entrusted to locally trained operators was the self-propelled scraper, the speed and weight of which made it very dangerous in the hands of a trainee. Early in the contract there were many delays through breakdown of the scrapers from rough treatment by the operators, but the contractor eventually countered this by introducing an effective and successful bonus scheme whereby the plant operators were rewarded if they kept their plant working for a month without breakdown from negligence.

Considering the terms of the contract, the amount of plant brought on to the job was considerable. All of it had suffered in transit and the efforts to bring it into commission, and then to maintain it in working order throughout the course of the job constituted a major item in the contractor's organization.

Construction Programme

The general programme of construction is indicated in Fig. 5, Plate 2.

The site was made available to the contractor from September 1952, but as a result of shipping difficulties and delays, and the difficulty in obtaining permission to transport plant through Syria, as well as the delays incurred in conveying plant along the hazardous route from Beirut to Amman, the contractor was not in a position to make a start until late in

November 1952. There was a further delay while the contractor was mainly occupied in training his local employees to operate and service his plant. All these cumulative delays were reflected in the slow rate of progress in the early months of the contract up to February/March 1953. From then on, however, the rate of progress improved considerably. In the first few months of the contract it had seemed doubtful if the time then lost could be made up, but in fact the contract was completed in less than 12 months against the contract period of 15 months. In that connexion it might be mentioned that, to achieve the required progress, the total labour strength on the job arose to a maximum of almost 2,500 against the contractor's original estimate of half that number.

Earthworks

The site selected was far from level and required considerable cut and fill to give the required gradient throughout the runway length.

Excavation was the biggest single item in the contract and amounted to 1·4 million cubic yards. Since there was a shortage of water for compaction purposes, it was fortunate that the natural moisture content of the soil was very close to the optimum moisture content. No water was added for soil compaction purposes. To take full advantage of the high natural moisture content of the soil, all excavated soft material was taken direct to the fill areas, spread, and rolled immediately with 8/10-ton smooth-wheeled rollers. The formation of spoil heaps or any double handling of excavated material was not permitted. In this way no difficulty was experienced in obtaining the minimum soil density of 95 per cent of the modified A.A.S.H.O. density.

Except under paved areas, the soil was deposited in 10-inch layers which compacted to 8 inches after consolidation. Each deposited layer was roughly graded with tractor-drawn scrapers, trimmed with a motor grader, and finally rolled with 8/10-ton smooth-wheeled rollers. Density tests were then carried out over the compacted layer ; at least one density test was made for every 1,000 square yards of fill area.

At first, the soil density was checked by the sand-bottle or core method but, because of the many large stones in the soil, these methods were unsatisfactory, and ultimately it was decided to use a much larger hole in the soil surface for density determination. The hole was lined with a rubber membrane (from a meteorological balloon) and the volume of the hole determined by filling with water. The average volume used on the site was $\frac{1}{2}$ cubic foot.

The hauls varied from a few feet to nearly $1\frac{1}{2}$ mile (see Fig. 4, Plate 1). Tractor-drawn scrapers were used for hauls up to 400 yards. The remainder of the excavation was carried out by self-propelled scrapers. Self-propelled scrapers employed consisted of four of 12/15 cubic yards, and six of 9/12 cubic yards capacity. Tractor-drawn scrapers consisted of four of 12/15 cubic yards, three of 9/12 cubic yards, and one of 6/9

TABLE 2.

Haul distance : mile	Average time in minutes per load With range in brackets			
	Self-propelled scraper Manufacturer's rating : cubic yards		Tractor-drawn scraper Manufacturer's rating : cubic yards	
	12—15	9—12	12—15	9—12
Nil—$\frac{1}{8}$	Not used	Not used	11·3 (5·0 to 13·5)	11·4 (10·2 to 13·6)
$\frac{1}{8}$—$\frac{1}{4}$	Not used	Not used	13·5 (10·0 to 16·5)	13·7 (11·7 to 17·7)
$\frac{1}{4}$—$\frac{1}{2}$	11·9 (9·7 to 14·5)	12·7 (8·65 to 15·6)	Not used	Not used
$\frac{1}{2}$—$\frac{3}{4}$	17·8 (14·7 to 20·8)	17·5 (14·5 to 20·3)	Not used	Not used
$\frac{3}{4}$—1	20·2 (19·3 to 24·0)	19·5 (17·0 to 22·7)	Not used	Not used
1—1$\frac{1}{4}$	25 (23·6 to 27·8)	Not used	Not used	Not used

cubic yards capacity. Average times taken by the scrapers, excluding the 6/9-cubic yard one, on the various hauls are given in Table 2. Detailed records of the output of each individual machine were maintained. Average output of all the scrapers for the whole of the contract for the day and night work was 35 cubic yards each per hour. During the day the self-propelled scrapers averaged 30·7 cubic yards per hour, and at night 35·2 cubic yards, with an overall average of 32·8 cubic yards. The tractor-drawn scrapers averaged 45·7 cubic yards per hour during daylight, and 42·2 cubic yards during the night, with an overall average of 43·6 cubic yards. On the average, 20 per cent more material was moved at night than during the day, mainly because shorter hauls were planned for night work to reduce the number of lighting sets required.

As has already been mentioned, nearly all the plant operators had to be trained on the job. The effect of this on scraper output was very noticeable from the records maintained, in that a skilled operator's output was about 33 per cent higher than that of the trainee.

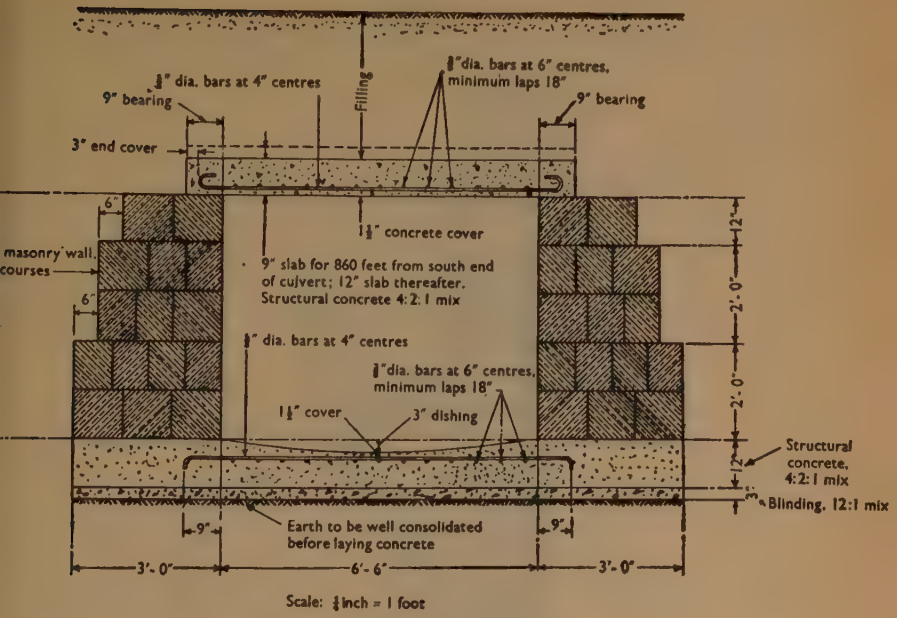
The output of the scrapers was also reduced by the general shortage of spare parts. Not long after the contract commenced, all the spares available in the Middle East had been used, and it took 3 to 4 months to obtain spares from England. Very urgent spares were, of course, obtained by air freight. While waiting for spare parts, machines often had to continue to operate where possible in their defective state at a reduced speed. In other cases machines were cannibalized. Tire consumption on the scrapers proved twice as high as expected. In that connexion it might be mentioned that the soil tended to become more stony with depth; the stones were angular, rough, and very destructive to plant tires. The average delivery time of new tires for scrapers was 7 months from England, and 5 months from the U.S.A. To reduce the effect of these delays a few machines were actually fitted with aircraft tires in place of the commercial type, but in the end this proved unsuccessful, since such tires had an extremely short life.

In the excavation area at the west end of the site, there was a large area of desert crust known locally as Nari. It consisted of a layer of calcareous deposit which varied in thickness from 3 inches to 18 inches; under the crust, there was a deposit of well-graded limestone gravel. The contractor had difficulty in removing this crust by rooter and scraper and finally had to resort to blasting to assist in its removal. The blasting was successfully achieved by using a charge of $\frac{1}{4}$ lb. to $\frac{1}{2}$ lb. of gelignite at depths ranging from 1 foot to 3 feet at 3-foot centres, firing 100 shots at a time.

Drainage

The site falls generally in a north-easterly direction. Features already mentioned are a range of hills on the south side and two wadis crossing the site. The larger of these wadis was replaced by a stone-and-reinforced-concrete culvert (*Fig. 6*).

Fig. 6



SECTION THROUGH MAIN CULVERT

The final strip was graded to fall 1 in 80 towards the east and 1 in 100 towards the north which made the drainage of the strip very simple. A main line of drainage was provided to the north of both the taxitrack and the runway, located 25 feet from the edge of the pavements in each case. A shallow channel was formed on the north edge of both the taxitrack and the runway with catchpits at 75-foot centres for the runways and 150 foot centres for the taxitracks. The catchpits discharged into the main drainage through 6-inch-diameter pipes (*Fig. 7*).

As a result of transport difficulties, breakages of earthenware pipes imported from the United Kingdom were very numerous, so that nearly all the pipes used had to be locally manufactured. Pipes over 6 inches in diameter were of spun concrete made in the Lebanon. All the 4-inch-diameter, and some of the 6-inch-diameter pipes were concrete pipes cast in situ with "Ductube." French drains were provided parallel to, and 12 feet south of, the taxitrack and runway. The pipes for these drains were of concrete, cast in situ with "Ductube"; the bottom half of the pipe was cast in dense concrete and the top half with ¾-inch "no-fines" concrete to provide a strong pipe with a pervious crown and an impervious invert (*Fig. 8*). The French drains were filled in with crushed stone in three layers with uniformly-graded limestone aggregate.

All the excavations for the culvert and drainage were dug by hand and

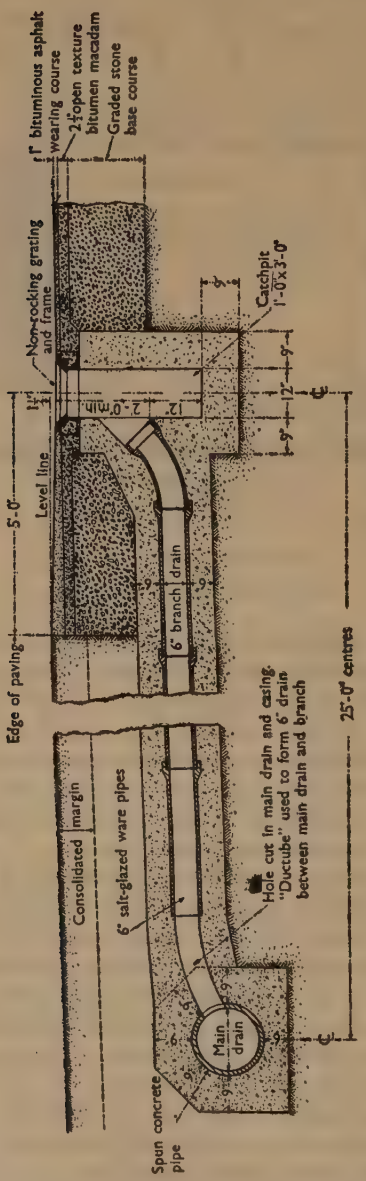
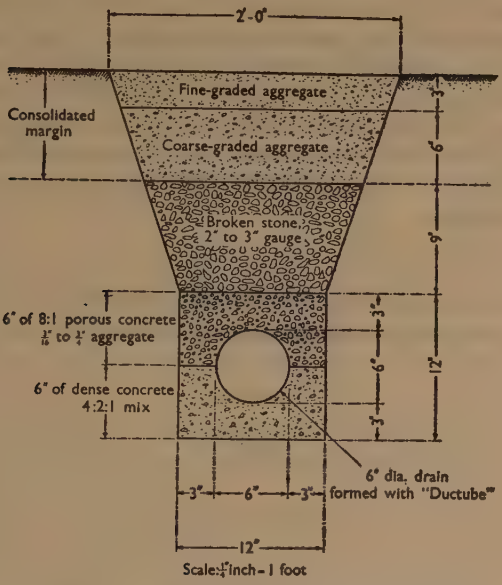


Fig. 7

TYPICAL SECTION THROUGH CATCHPIT AND MAIN DRAINAGE, SHOWING PAVEMENT CONSTRUCTION

without shuttering. The deepest drain was 16 feet deep. In no case did the earth walls of the trenches collapse.

Fig. 8



Typical Section through French Drain

Hard Filling

Where the existing levels under paved surfaces were below the required formation level, the foundation of the pavement was formed with hard filling. Consolidated margins were provided on each side of the runway, taxitrack, and concrete platforms. The runway margins were 75 feet wide strengthened with a 9-inch layer of hard filling and the taxitrack and platform margins 30 feet wide with a 12-inch layer of hard filling.

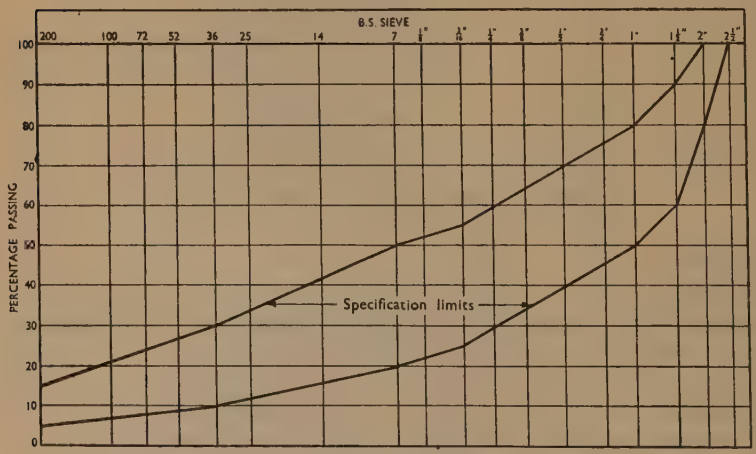
Two types of hard filling were used: the first, a naturally occurring limestone gravel which was found in small quantities at the west end of the runway strip; the second, hand-broken limestone mixed with clay as recommended by the site investigation team. The silty limestone gravel actually contained slightly more material passing a No. 200 B.S. sieve than the range of 9 to 15 per cent allowed in the specification. Before it could be approved, tests were made to check its suitability, and it was found to provide a good surface and to be capable of carrying traffic without rutting. Permission was given for this material to be used for the consolidated margins, and hand-broken limestone for fill under paved areas.

The hand-broken stone was mixed with clay and was formed into a hard filling by spreading a 5-inch layer of clay and covering the clay with a 5-inch layer of stone, then rolling the stone into the clay with 10/12-ton smooth wheeled rollers. The stone was spread evenly over the clay with scrapers in the first instance and picked over by hand to form a uniform layer before rolling. The process was repeated to provide additional layers as required. This fill proved to be quite stable and was exposed to very heavy continuous rain during March 1953 without showing any sign of instability. Although crushed stone would have been a better material for hard filling, the hand-broken stone was adequate for the job. It was considerably cheaper than the crushed stone and, in addition, provided more men with work.

Base Course

Since there was very little available material intermediate in strength between clay and rock it was necessary to construct the whole of the

Fig. 9



Grading Curves for Crushed-Stone Base-Course Aggregate

flexible pavement with graded crushed stone. Large quantities of this material were required ; the grading limits for base course aggregate were as shown at *Fig. 9*. In spite of real effort it was found difficult to produce a crusher-run aggregate within these limits. Usually, the crusher-run aggregates were deficient in fines. There were jaw-and-hammer crushers available, and it was found that the hammer crushers produced the better gradings. Considerable time was spent experimenting with the crushers to try and produce more fines with aggregates of less than $2\frac{1}{2}$ inches.

Baffles were introduced into the jaw crushers and some of the openings on the hammer-crusher grids were reduced in area. Although a few of the crushers were modified in this way successfully, the bulk of the base course aggregate was, in fact, produced from a mixture of fine and coarse aggregate.

The base course was laid in three layers on the runway and four layers on the taxitrack. The first layer of base course was crushed and mixed on the pavement formation by transportable crushers, thereby saving double handling. It was then spread by tractor-drawn scrapers. To avoid damage by plant to the first layer, this method was not adopted for successive layers. The remainder of the base course was, therefore, either crushed at the quarries or elsewhere on the site and transported to the pavement areas by means of tipper lorries. Here it was roughly spread by tractor-drawn scrapers in layers not exceeding $2\frac{1}{2}$ times the maximum aggregate size, and graded ready for the rollers with a motor-grader. It was then watered at the rate of 4 gallons per square yard and rolled with an 8/10-ton smooth-wheeled roller until the required density was achieved. Since during the course of spreading and watering, the fines tended to drop to the bottom of the layers it was necessary to cover the surface with $\frac{1}{8}$ inch down aggregate after initial rolling to fill all the voids and obtain a thoroughly dense layer.

The final layer of base course was spread very carefully. Small concrete blocks were boned into the exact finished level of the base course at 10-foot centres and screeding to these blocks produced a surface on the base course that was almost within the specification requirements for the finished asphalt surface. The care taken in placing this layer was rewarded in simplifying the laying of the bitumen macadam and in reducing to a minimum the running adjustments to the asphalt finisher.

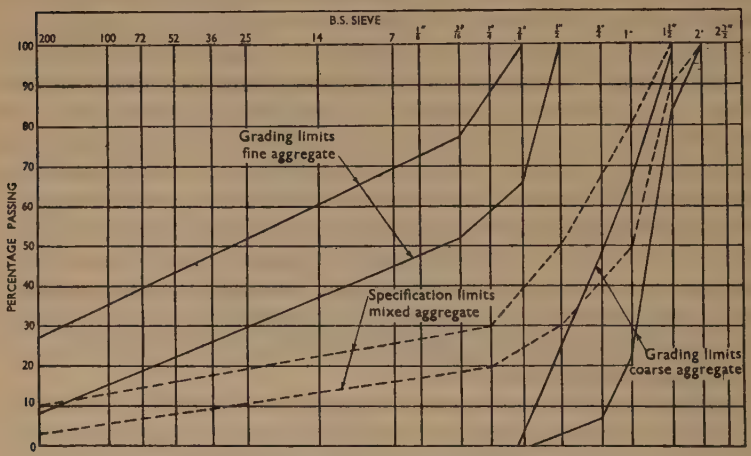
Bituminous Surfaces

The bituminous surface consisted of a 1-inch-thick bituminous-asphalt wearing course on a $2\frac{1}{2}$-inch open-graded bitumen-macadam base course. The binder was straight-run bitumen 60/70 in the taxitrack and 80/100 in the runway. Both coats were of course laid hot. The base coat was manufactured with coarse and fine aggregate mixed by volume in the ratio of 7 parts of coarse to 4 parts of fine aggregate and bound with 3·5 per cent (by weight) of bitumen. The gradings of the two aggregates together with the grading of the combined aggregate are shown in *Fig. 10*. This specification produced a dense and impervious base coat of good stability (*Fig. 11*, facing p. 408).

The asphalt wearing coat was mixed by weight in the ratio of 45 per cent coarse aggregate to 55 per cent fine aggregate and bound with 6·3 per cent of bitumen. No filler was added. The gradings of the coarse and fine aggregates together with the combined aggregate are shown in *Fig. 12*. In the manufacture of the asphalt, approximately 10 per cent of

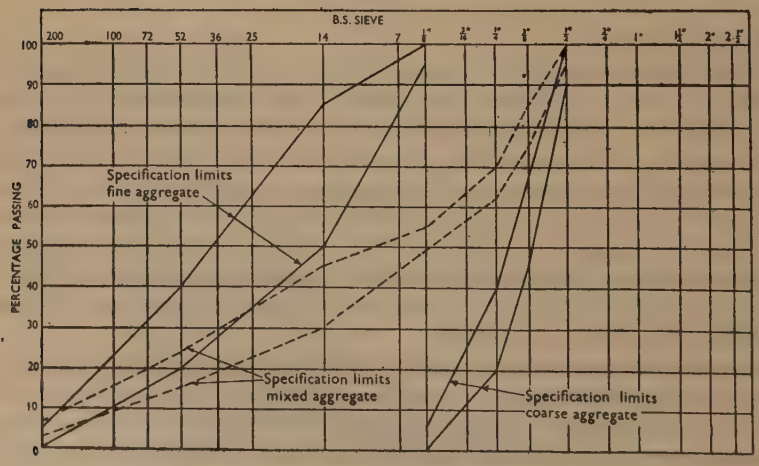
the aggregate was blown out of the mix by the dryer and the bitumen content was adjusted continuously on the results of hot extraction tests. The high quality crushed stone aggregate, in the proportion given, produced a wearing surface that had the stability required to resist the shear stresses set up by the high tire pressures of modern aircraft and gave a

Fig. 10



GRADING CURVES FOR BITUMINOUS MACADAM (OPEN-TEXTURE) BASE COURSE AGGREGATE

Fig. 12



GRADING LIMITS FOR ASPHALT WEARING COURSE AGGREGATE

surface that was sufficiently rough to prevent skidding but not so harsh as to damage aircraft tires (see *Figs 13* and *14*, between pp. 408 and 409).

The bitumen mixing plant was set up in a central position on the north side of the strip. It consisted of three Miller's highway mixers which were used as dryers only and three Miller's model EE mixers. The dryers were set up to discharge directly into the skips of the mixers. The plant was located in front of one 300 gallon bitumen storage tank which supplied all the mixers with hot bitumen (see *Fig. 15*). The tank was filled from three 600-gallon tar boilers and three 600-gallon tar sprayers. The pipe lines from the storage tank to the mixers were arranged so that at the end of the day bitumen drained into the storage tank and the mixer, leaving no bitumen to solidify in the delivery pipes. The three mixers were mounted in line and high enough to permit tipper lorries to pass under the mixers. The lorries passed under the three mixers in turn until they received a full load.

The specification required all materials for the wearing coat to be mixed by weight, and the mixers and dryers were modified locally to meet this requirement. On the model EE mixer, ordinary platform scales of local manufacture were mounted to support the bitumen trough, and the weight of bitumen in the trough was controlled and checked frequently. The aggregates were weighed in locally manufactured timber hoppers which were mounted on platform scales. The hoppers were loaded by hand and discharged directly into the skips of the dryers. The output of the mixing plant varied with the day temperature and averaged 5¼ tons per hour per machine ; with three machines working for 11 hours a day, the daily average output of the plant was 173 tons.

The bituminous mixture was delivered in steel-bodied tippers to a Barber-Greene Olding junior finisher. This finisher was able to lay at a speed that suited the mixer output when operated on the first speed of the low operating range which was measured at 8 feet per minute. Occasionally the machine was operated at the second speed at midday when there was a higher output from the mixer owing to the shorter period required to heat the aggregates. The maximum laying width of the finisher was 8 feet 6 inches but here the width laid was 8 feet 3 inches which allowed a 3-inch overlap for matching joints. Cut-off plates were used to produce narrower strips when breaking joint between the bituminous base course and asphalt.

Both the bituminous macadam and the asphalt mixes were very stable in the hot condition and it was possible to compact the carpet with a 6-ton tandem roller immediately behind the finisher. The 6-ton roller was followed by a similar roller and a 10-ton back roller. The asphalt was laid from 1⅛ to 1¼ inch thick to compact to one inch and the "bitmac" from 2⅞ to 3 inches thick to compact to 2½ inches after rolling.

The temperature of the bitumen before entering the mixer ranged from

325° to 350° F., the aggregate leaving the dryer was from 300° to 350° F., and the mix delivered to the finisher was from 300° to 325° F.

The mixers were manned by 14 operators and 114 labourers, and the finisher by 4 operators and 16 labourers. In addition, there were 8 lorry drivers bringing the mix from the mixer to the finisher and four roller drivers, making a total of 160 men altogether.

Concrete Pavements

The double-slab concrete pavement was constructed at each end of the runway to provide running-up platforms for jet aircraft. The upper slab was 8 inches thick and the lower slab 9 inches thick. The lower slab was constructed in bays 11 feet 6 inches wide with expansion joints every 126 feet 6 inches ; the upper slab was laid in 11-foot-6-inch-wide bays with dummy joints every 11 feet 6 inches and an expansion joint every 57 feet 6 inches. The joints in the upper slab were placed not less than 2 feet from the joints in the lower slab. To ensure that there was no bonding between the upper and lower slab, the two slabs were separated by bitumen emulsion, laid at the rate of 3 square yards to the gallon, and a layer of building paper laid on top of the bitumen emulsion.

Over the aircraft standing areas the joints in the top concrete slab were sealed with a compound wholly resistant to the solvent action of kerosene. This joint sealer is not heat resisting but it does harden more rapidly than normal joint sealers after softening by heat. The remaining joints of the upper slab were sealed with a less costly sealer which is reasonably resistant to heat but not resistant to kerosene. In order to minimize the effect of jet exhaust on the joint sealers, the sealing compound was kept $\frac{1}{4}$ inch below the finished concrete level.

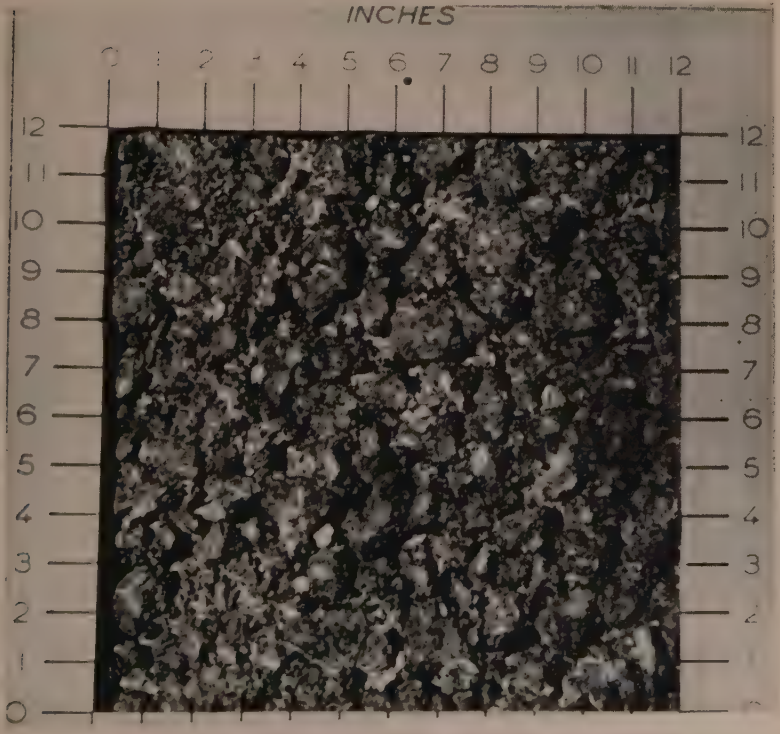
Both the upper and lower slabs were constructed of high-grade-quality concrete and the concrete produced an average crushing strength of 4,750 lb. per square inch at 28 days against the specification requirement of 3,500 lb. per square inch minimum at 28 days. It will be noted from *Fig. 16* that although the fine aggregate had an excess of material passing the number 200 B.S. sieve, this apparently had no effect on the strength of the concrete. Initially, the water content of the concrete appeared to be high by normal standards. This was because the aggregate was very dry and absorbed water equal to just over 1 per cent of its weight and the hot dry winds took some of the water out of the mix by evaporation. Although the water added gave a water/cement ratio of 0·57 it was considered that the final water/cement ratio was just less than 0·50.

The lower slab was cured by a bituminous membrane of bituminous emulsion sprayed on to the fresh concrete at the rate of one gallon per 10 square yards, additional protection being given to the slab with wet rush mats for the first 24 hours of curing. Membrane curing was not allowed on the upper slab because a suitable membrane-curing compound was not available for concrete pavements that were to be subjected to jet

Fig. 3

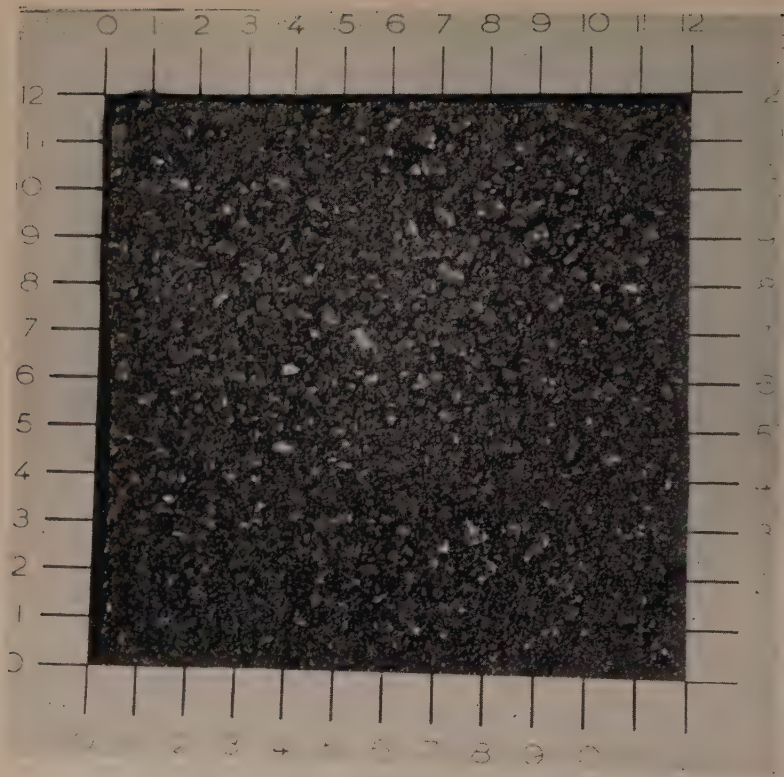
MERKA VILLAGE SITE SHOWING EXISTING CAVES (TO BE FILLED) AND VIEW OF EASTERN APPROACHES TO SITE

Fig. 11



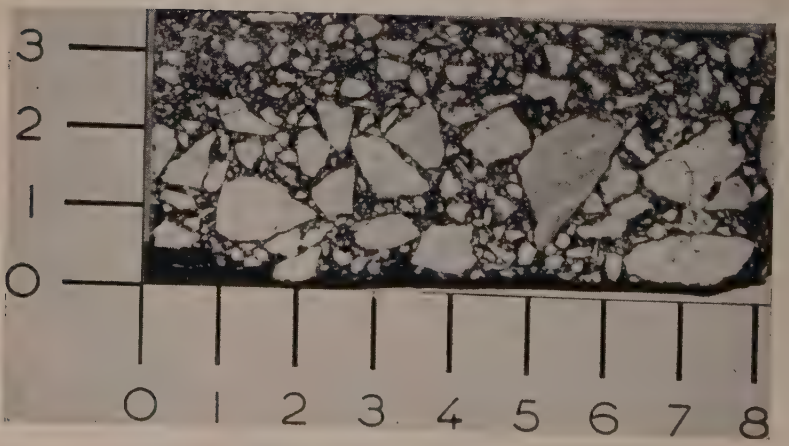
TEXTURE OF BITUMINOUS MACADAM SURFACE

Fig. 13



TEXTURE OF ASPHALT WEARING SURFACE

Fig. 14



SECTION CUT THROUGH BITUMINOUS CARPET AS LAID WITH 2½-INCH "BITMAC" WITH 1-INCH WEARING CARPET. THE FIGURES ARE INCHES

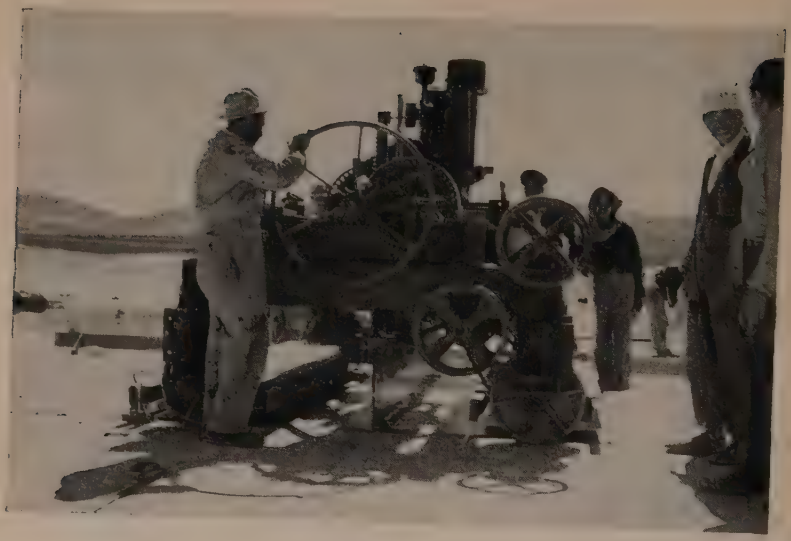
Fig. 15

General View of Bitumen Mixing Plant

Fig. 17

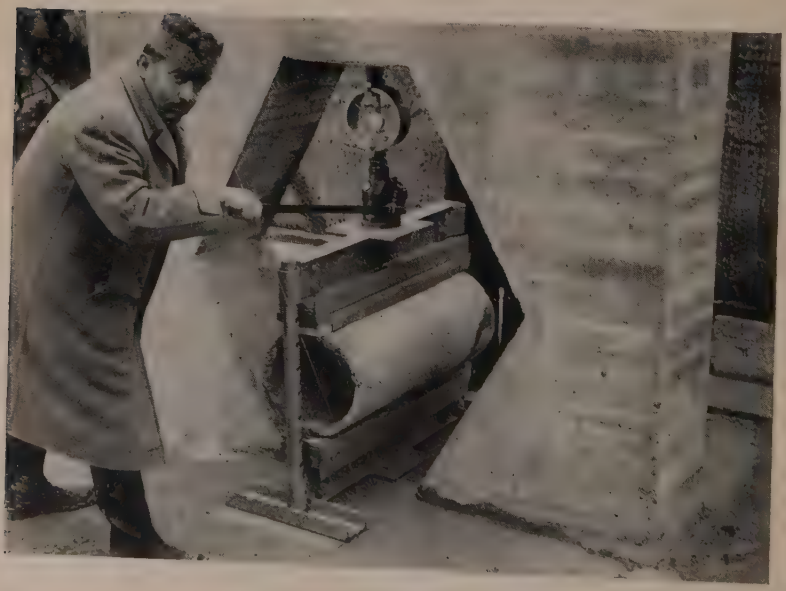
Section of Concrete cut from Pavement

Fig. 18



Vibrating Joint Former for Concrete Pavements

Fig. 20

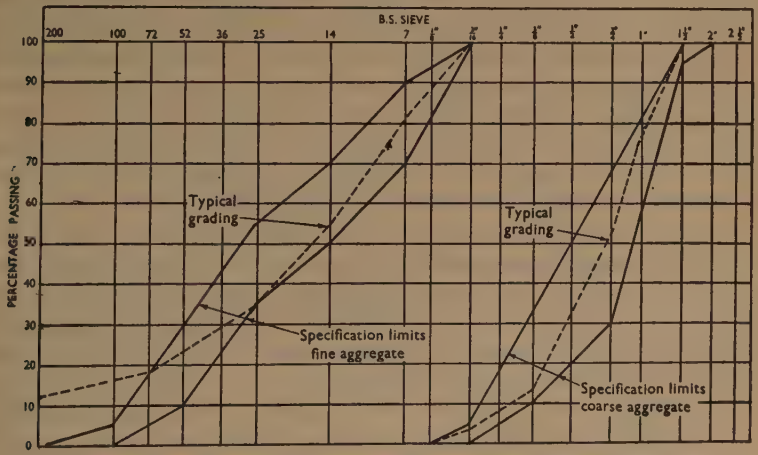


Crushing Test on Storm-water Drainage Pipe

blast. The upper slab had therefore to be cured by the normal method of keeping the fresh concrete surface wet for the first 7 days. The slab surface was kept wet with close-woven rush mats. These mats proved to be a very effective curing agent, equal perhaps to ponding. They kept a film of water over the concrete surface from $\frac{1}{4}$ inch to $\frac{1}{8}$ inch in depth.

The concrete was mixed at a central point on the site in four 2-yard and one 4-yard American truck mixers. These mixers were not satisfactory. The 2-yard mixers were charged from weigh batchers and the 4-yard mixer was charged from tipper lorries which were calibrated by volume. The truck mixers discharged into tipper lorries which delivered the concrete to the site. When the truck mixers were brought into use on the site, the

Fig. 16



GRADING LIMITS FOR CONCRETE AGGREGATE

drums revolved at 9 revolutions and 4 revolutions per minute for the 2- and 4-yard mixers respectively, but it was soon found necessary to increase the number of revolutions per minute. Even after this modification the mixers were not entirely satisfactory. They did not discharge cleanly, possibly because they had been designed for a wetter mix. The 2-yard mixers produced only $3\frac{1}{2}$ mixes per hour taking $2\frac{1}{2}$ minutes to load, $5\frac{1}{2}$ minutes to mix, and $8\frac{1}{2}$ minutes to discharge. The 4-yard mixer produced $2\frac{3}{4}$ mixes per hour, taking 5 minutes to load, 11 minutes to mix, and 7 minutes to discharge. All the mixers were loaded to three-quarter capacity. Eighty-two men were employed supplying the concrete mixers which produced altogether only about 190 cubic yards of concrete per day.

The greater proportion of the concrete pavement was laid with concrete paving machines which ran on 8-inch steel side forms. The concrete was

spread with a trough-type distributor which deposited the concrete evenly between, and as much as 2 inches higher than, the forms. The spreader was followed by a tamping and finishing machine which cut off the concrete with a revolving blade 1·4 inches higher than the forms. The machine was passed over the concrete twice, first with the vibrating tamper set 0·8 inch higher than the forms and finally with the tamper set down to the exact slab thickness. These settings were obtained by experiment and produced a very dense slab of concrete (*Fig. 17*). The machine was fitted with an oscillating screed at the back of the machine which rested on the side forms and produced the final finish to the slab. The best speed obtained for the tamping and finishing machine was 4 feet 8 inches per minute. The concrete at the edges of the slab was additionally vibrated with poker vibrators. The paving machine was followed by a vibrating joint-former. This machine formed all the transverse expansion and dummy joints while the concrete was still wet. The expansion-joint filler was put into position as soon as the joint was cut. It was pressed into the concrete by hand until flush with the surface and driven on to the correct depth with the joint "cutter." This method of forming the joints resulted, of course, in the displaced concrete forming a slight bulge on either side of the joint. These bulges had to be removed by hand. Rounded edges were formed at the joints with specially made trowels (see *Fig. 18*).

The concrete slab was laid to a fall of 1 in 80 longitudinally and 1 in 100 transversely making a maximum "dip" of 1 in 63. It was found that, even at this small gradient, under vibration the concrete flowed to the lower side of the slab, making it difficult to obtain flush joints, and that concreting with the fall the slab developed cracks at about 2-foot centres at right angles to the "dip" of the slab.

The concrete paving machines were manned by fifty-three tradesmen and labourers.

Narrow and curved lanes were concreted by hand and tamped with a vibrating plank.

Testing Materials

A small laboratory was established on the site for day-to-day checks on the quality and strength of materials. The laboratory was equipped to make the following tests :—

(1) Borings and moisture content.
(2) Modified A.A.S.H.O. tests for maximum dry density of soils.
(3) Field density tests (Fig. 19, Plate 2).
(4) Atterberg limits.
(5) Sieve-analysis.
(6) Percentage of bitumen—hot extractor method.
(7) Concrete-beam strength.
(8) Pipes—crushing strength.

(9) Aggregate crushing strength.
(10) Temperatures of aggregates, bitumen, and bitumen mixtures.
(11) Plate bearing tests.
(12) Rubber-tired roller tests.

To facilitate crushing tests a reinforced-concrete frame was built that would withstand a load of 50 tons (*Fig. 20*, facing p. 409).

The field density test was made by the water method, that is to say measuring the volume of water required to fill the hole, which was lined with a rubber membrane to prevent water from seeping into the surrounding soil.

The laboratory work and field tests were carried out by locally engaged assistants who, for the most part, had no previous knowledge of any such work and had to be trained through all stages. The local staff, however, proved quick and willing to learn and it was possible, after a comparatively short time, to entrust all routine tests to them with only the minimum of supervision.

The 7-day-concrete 6-inch-cube tests for the pavement concrete were made by a local cement factory. Unfortunately, the crushing machine at the factory was designed for small cement-testing cubes and could only apply a load to the 6-inch cubes that produced just over the specification strength of 2,250 lb. per square inch.

The 28-day-concrete 6-inch-cube tests for the pavement concrete were made at the A.M.D.G.W. Laboratory, Habbaniya, Iraq.

The cement and bitumen were analysed at the A.M.D.G.W. Laboratory, Cardington, Bedfordshire.

The flexural strength of the pavement concrete was determined from beam tests made at the site laboratory. Beams 36-inches-by-12-inches-by-8-inches deep were loaded at third point for a span of 30 inches. The 7-day flexural strength of these beams averaged 646 lb. per square inch.

Extensive plate-bearing tests were carried out throughout the period of construction, but the analysis of all these results had not been completed at the time of writing the Paper.

General

The runway has now been completed and it is sufficient comment at this stage to say that all the tests and examinations carried out indicate that the runway has been constructed up to the full requirements of the specification in all respects. Whilst this was of course the whole aim and purpose throughout the contract, it will be remembered that it was achieved in spite of local inexperience in such work and in spite of the many difficulties which arose from the location of the site.

Rigid control and supervision through all stages of the contract and strict adherence to the requirements of the specification were maintained from the outset and their aim has been achieved.

In so far as the finished surface of the runway is concerned, it might be mentioned that the specification required all finished surfaces to show no deviation exceeding $\frac{1}{4}$ inch when tested with a 10-foot straight-edge and that the results obtained were well within the specification—generally the deviation was less than $\frac{1}{8}$ inch.

The people of Jordan have shown themselves quick and ready to learn all aspects of such work as was covered in the Amman contract and the experience so gained by local labour should prove of value for other large civil engineering schemes which are now being planned for Jordan.

Acknowledgements

The Paper is presented by permission of Sir G. H. Fretwell, K.B.E., C.B., M.I.C.E., Director-General of Works, Air Ministry.

The Authors wish to thank Mr A. J. Michie, A.M.I.C.E., and Mr P. Kent, A.R.I.C.S., for their assistance in the preparation of the Paper.

Discussion

The Authors introduced the Paper with a series of lantern slides and a film.

Mr F. S. Snow considered that the construction difficulties had been understated, particularly the relations between the Arab and British personnel. Mr Snow had visited the site on a number of occasions during the progress of the work and had marvelled at the patience shown by the Resident Engineer, Squadron Leader Smith, and by his second-in-command, Flight Lieutenant Michie, and at their skill in overcoming the problems of supervision and testing.

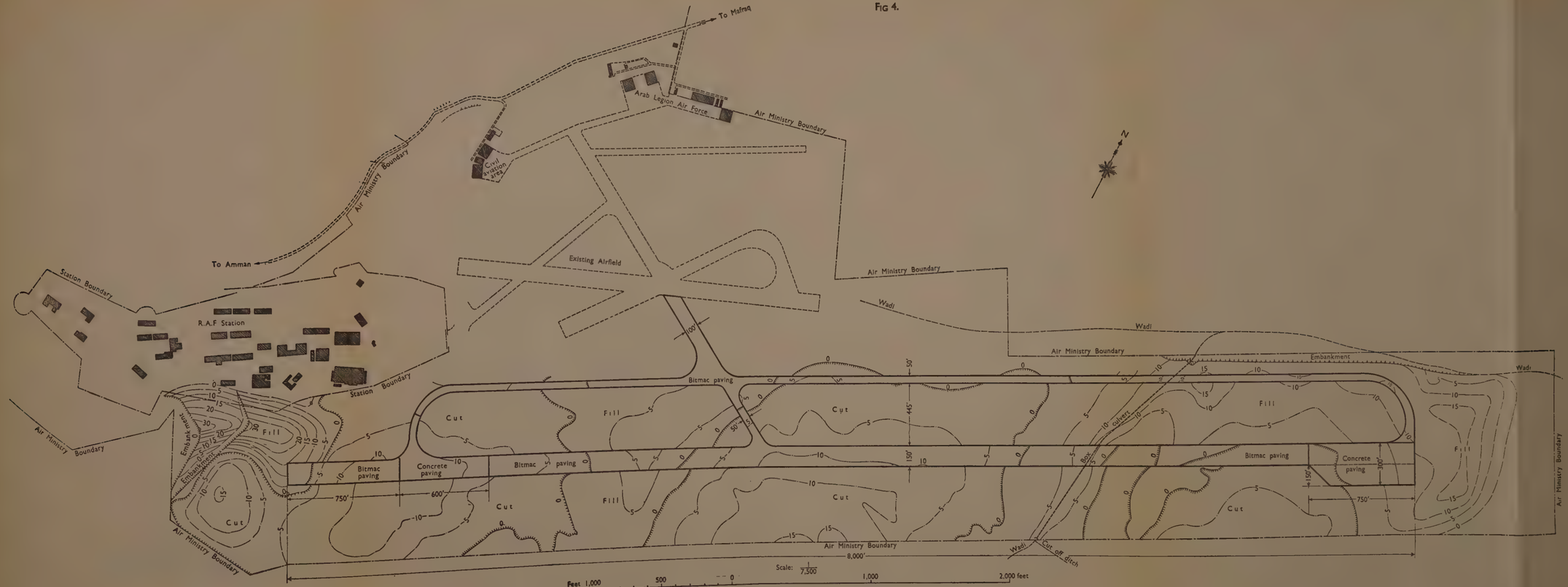
To those unfamiliar with the Middle East, the Paper did not stress sufficiently that Amman airport had been constructed in a very bare and desolate country. The terrain could be likened to the English Lake District, without the Lakes. The problems, therefore, of site selection, survey, and design had been considerable, and the successful execution of the contract had been a very great British and Arab achievement.

Having had some experience in England in the removal of cemeteries, particularly in the City of London, with the attendant difficulties with ecclesiastical authorities, archaeologists, the city engineer and sanitary inspector, and the descendants of the deceased, Mr Snow sympathized with the Authors' difficulties in removing more than nine hundred bodies from the Moslem cemetery at Amman.

What difficulties had been experienced in the runway design because of the height above sea level, the temperature, and the wind directions ? Those factors must have affected the length of runway required consider-

THE CONSTRUCTION OF A NEW RUNWAY AT AMMAN, JORDAN

FIG 4.



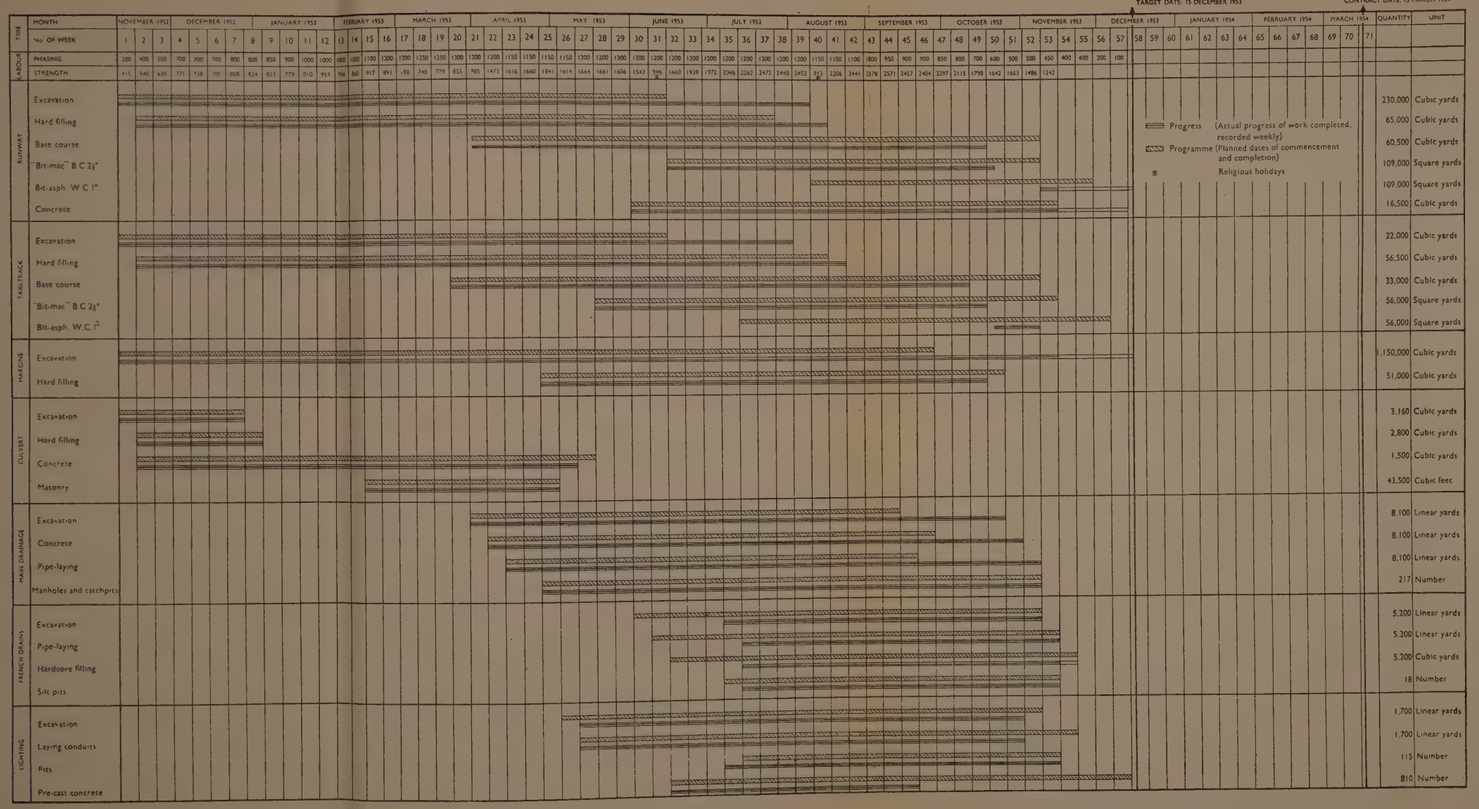
LAYOUT OF EXISTING AIRFIELD AND NEW RUNWAY SHOWING CUT-AND-FILL AREAS

A. B. MACRAE AND A. F. SMITH

PLATE 2
NEW RUNWAY AT AMMAN

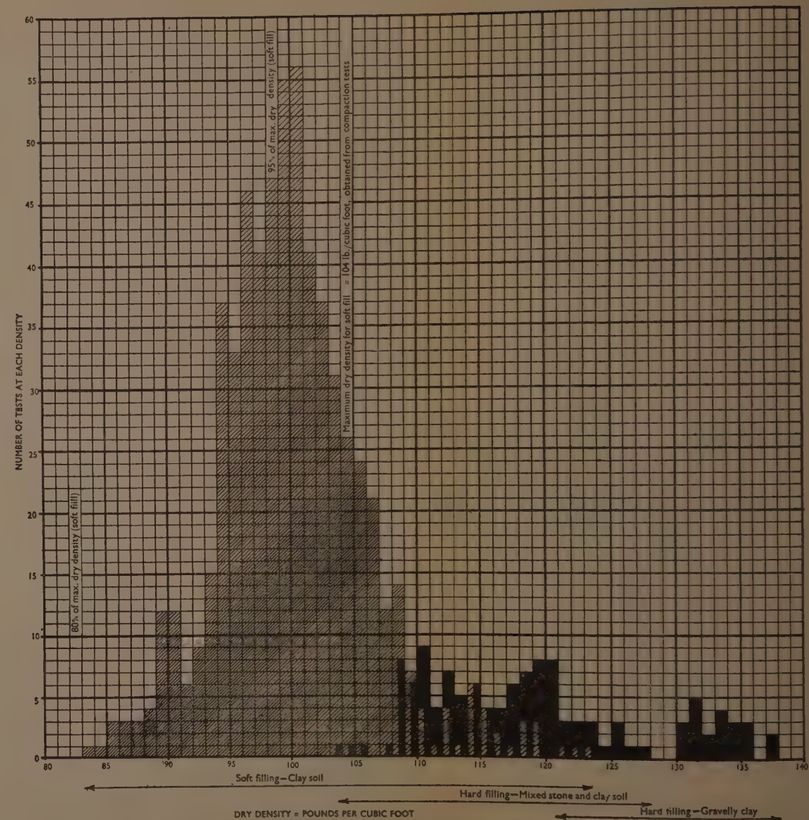
THE CONSTRUCTION OF A NEW RUNWAY AT AMMAN, JORDAN

FIG. 5



PROGRAMME AND PROGRESS CHART

FIG. 19


NUMBER OF TESTS AT EACH DENSITY
80% of max. dry density (soft fill)
95% of max. dry density (soft fill)
Maximum dry density for soft fill = 104 lb. cubic foot, obtained from compaction tests
Soft filling—Clay soil
Hard filling—Mixed stone and clay soil
Hard filling—Gravelly clay
DRY DENSITY = POUNDS PER CUBIC FOOT

Required dry densities for soft filling ranges from 95% of the maximum dry density at a depth of 6" to not less than 80% of the maximum dry density at a depth of 24"

ably, including the lengths of stop-way and clear-way necessary for civil aviation.

Referring to the question of water supply, Mr Snow said Jordan was desperately short of water. How had the Authors managed to obtain sufficient water for mixing and curing the concrete, and other purposes ?

Since much of the fill had been carried out with hand-broken stone, it would be interesting to learn whether the Authors had experienced the same difficulties with the grading of the hardfill material as had been encountered in the construction of the airport at Kalandia, Jerusalem. It was common practice in Jordan to construct roads with hand-packed broken stone, in the same way that hardcore was used in the United Kingdom, but in Jordan the voids and interstices were not filled with fine material. That type of construction had the happy knack of standing up to the heaviest rollers without deflexion, but did not meet the required specification as to density.

Deep trenches had been excavated without timbering. In the United Kingdom the problems of liability and insurance would have been in the forefront of the minds of those responsible. Information on that subject, and also on the wage problems which had existed on the contract would be welcomed.

It appeared from the Paper that the whole of the drawings and information supplied to the contractors had been in English dimensions. Mr Snow's firm had carried out work in Jordan for the Hashemite Kingdom, and had been compelled to translate all measurements into the metric system !

Why had it been decided to lay the concrete pavement as a sandwich construction with staggered joints and to a total thickness of 17 inches ?

The de Havilland Aircraft Company's runway at Hatfield had also been based on the Westergaard theory, but was only 10 inches thick on ground with a k-value of 300 lb. per square inch per inch. It had been proved by extensive tests [1] that the type of construction used at Hatfield was superior to that used at Amman.

If core tests had been taken of the actual concrete work at Amman, the average cube test figure of 4,750 lb. per square inch might well have been exceeded.

The Authors had described in the Paper the method of cutting the warp joints, and the way in which the displaced concrete had formed a bulge which had had to be removed by hand. That was a very big problem in both road and runway construction. Mr Snow considered that to disturb in any way concrete which had been so thoroughly compacted must necessarily be harmful. Would the Authors give their views on that ?

Since the new runway had been planned to meet civil needs as well as those of the R.A.F., Mr Snow showed a lantern slide which illustrated a

[1] F. S. Snow, "The de Havilland Airfield." Airport Paper No. 12, Instn Civ. Engrs, 1949.

model of the civil area being designed for the Hashemite Kingdom on the actual site. The civil area had been designed not only to deal with the local airlines, but for use in connexion with transit aircraft on the London–Baghdad route.

A second slide showed the design for the main airport building. It was intended to allow for extensions, because Amman was looked on as an international stopping-place, as distinct from Jerusalem, which was for tourist traffic. The building was to be in reinforced concrete, stone faced, stone being a material used throughout the whole of Jordan. The local contractors knew stonework from A to Z and obtained very pleasing results from their own natural materials. It was intended to use the white stone of Amman for the terminal buildings, offices, etc., and to transport from the quarries at Jerusalem the pinkish/reddish stone for the central control tower.

Group-Capt. E. J. Palmer, Commander of the R.A.F. Station at Amman while the construction of the new runway had been in progress, observed that, under the terms of the Anglo-Jordan Treaty of Alliance signed in 1948, the Royal Air Force had been invited to maintain units at Amman and Mafraq, and although there were other sites within about 20 miles of Amman which would have been first-class, they were unsuitable for development as an R.A.F. base.

The hills to the south ran parallel with the runway, and at the western end there was a very sharp fall to a valley, but farther to the west there was a large hill, the Jebel Ja'far, which encroached to the northward and was in fact in the 15-degree funnel at the western end of the runway. In addition to that, at the eastern end of the runway and outside the Air Ministry boundaries a low ridge which ran roughly from south to north presented a tricky problem from the approach point of view. It had been necessary, owing to the elevation above sea level of 2,500 feet, to get the maximum possible length of runway, whilst of course preserving reasonable approach angles, and also some length of over-run.

If the runway were moved to the west it would ease the approach problem from the east, but it would cut down the over-run at the western end ; and the over-run at the western end, from a pilot's point of view, was a difficult one, because if a plane went too far it plunged into the valley down a slope considerably steeper than 1 in 2. On the other hand, moving the runway to the east in order to extend the over-run at the west raised the approach problem caused by the ridge which ran across from south to north. There were possibilities of moving the centre-line north or south, but movement to the south brought the hill at the western end, the Jebel Ja'far, too far into the funnel to be acceptable, and movement of the centre-line to the north brought it too close to the camp and to the transmitting-station aerials at the western end of the camp. The question of the actual alignment of the runway had been a difficult one. By increasing the orientation it was possible to get clear, or almost clear,

of the Jebel Ja'far in the western funnel, but it meant that the eastern funnel was encroached into by the range of hills which ran to the south of the centre-line.

Engineers would be able to appreciate, therefore, all that had been involved in the original survey. There had been at least three complete surveys before the position of the centre-line had been finally settled. It had involved acceptance of the Jebel Ja'far encroaching into the western funnel, but it gave an acceptable approach from the east, which was the normal landing direction, and it did not present any great hazard in the western funnel.

Referring to the question of approach lighting, Group-Capt. Palmer said that approach lighting in the eastern funnel was reasonably easy, and straight-line or any similar modern type of approach lighting could be installed. At the western end, however, because of the very steep fall to the valley, there could be no question of erecting poles for centre-line lighting. The other problem concerned with the lighting was the range of hills to the south. Within the rules of the Air Traffic Control organization, the whole line of hills called for obstruction lighting. That presented a great problem in the supply of power and also in the laying of cable.

Flying aids for bad weather presented a very difficult problem, again because of the siting and the surrounding hills. Although the weather throughout the year was generally good, during the months of February and March in particular the cloud was often right down on the surrounding hills, giving a very small clearance underneath, and it was extremely difficult country in which to do a blind let-down. As would be appreciated, the installation of radio aids was usually very badly affected by surrounding hills. That problem would be extremely difficult in bad flying conditions.

Nevertheless, in spite of the difficulties, both from the engineering and the sighting points of view, an excellent result had been achieved. The new runway at Amman was a very valuable addition to the aviation facilities in Jordan.

Mr Emile Bustani observed that despite the earlier conclusion on p. 393 that it was not possible to construct a runway to ideal standards, especially with regard to specification, on p. 29 the Authors testified that first-class adherence to specification had been maintained and very good results obtained.

As an example of the difficulties encountered, there had been demonstrations against the British, and therefore against the contractor, by Communist-inspired gangs, and as a result the contractor had been ordered by the Government to pay higher wages than the Government paid to labour. That had been a comparatively minor difficulty, but it had caused tension. Mr Bustani had told their engineers not to pay the higher wages demanded and as a result the engineers had had to keep within their camp for 2 or 3 weeks, because they had been in danger of being shot. There had been many other difficulties of that kind which had been surmounted.

One accident which occurred might have been disastrous, but for luck, or what the Arabs called " kismet." Some of the wonderful pits and caves which had been left by the Turks many years ago were shown in *Fig. 3*. Towards the end of the job, on one of the taxiroads round the runway—not on the runway itself—a grader had suddenly fallen, driver and all, into a pit which had been covered by only a foot of earth. Many D8 12-cubic-yard scrapers and D7 rollers had passed over that same road, and even the cars of H.R.H. the Duke of Gloucester and H.E. the President of the Lebanon, and nothing had happened !

Some of the twelve Britons who had helped to train the local men to become good plant operators had worked with the contractors before on a 30-foot pipeline and were used to dealing with Arab labour and were happy to take part in the job at Amman. Some Members of Parliament from the United Kingdom had visited the Middle East last year and called at the Amman airport. All those who had seen the work at Amman had testified to the fact that the local boys learnt well and quickly.

Mr K. C. Mann said that it was one thing to design an airfield in London, applying the techniques which had been developed through past experience, but it was another matter to translate that theory into practice in remote countries, often, as in the case of Jordan, difficult of access, at any rate from the sea. At the design end of the job the prospects of building a modern airfield under the conditions then prevailing in Jordan had not been encouraging, since so far as could be seen the principal asset appeared to be the very large pool of unskilled labour, derived mainly from the Arab exodus from Israel. It was, of course, the natural desire of the Jordan Government, who had the refugees on their hands, to obtain employment for as many of them as possible.

The Jordan Government had limited the contractor to a maximum of 30 people brought into the country from outside, including the contractor's agent and the supervisory staff. Mr Mann understood that at one time the Jordan Government objected to the use of tipping lorries, because of the reduced demand for labour which resulted. Nevertheless, a good balance appeared to have been achieved between the use of mechanical plant and the use of unskilled labour. The contractor had been faced with the task of training men to operate plant, all the important items of which had to be imported into Jordan specially for the job.

On p. 390 the Authors referred to the digging of pits at 600-foot centres each way over the site to investigate the soil conditions. Had much variation been found in soil conditions in the intervening spaces, because it was always a difficult matter to decide the extent to which trial pits should be dug on a preliminary investigation ? A significant factor when considering the working value of the California Bearing Ratio to be adopted was illustrated in *Figs 2*, which showed that compaction did not increase the value of the C.B.R.

In that connexion it was interesting to see that the mechanically

stabilized base course to which the Authors referred on pp. 404–405 had been achieved by the use of correctly graded material without recourse to imported, manufactured, or what the Americans called "end" items, such as cement. The importation of such materials into a country such as Jordan would have been a very costly matter. It also showed that by the use of carefully graded stone it was possible to achieve a very dense and accurately finished surface. The finished surface had enabled the asphalt spreader to work with little adjustment and had avoided the use of additional pre-mixed base-course asphalt to iron out irregularities. Often surface irregularities involved laying macadam in quantities far in excess of the estimated requirements and not infrequently led to arguments with contractors over the quantity of black top that should have been required if the formation had been well prepared.

It was interesting to see from p. 408 that the specification requirement for the strength of the concrete was 3,500 lb. per square inch minimum at 28 days, and the contractor had achieved an average of 4,750 lb. per square inch. In designing slabs it was necessary to assume that a compressive strength would be obtained which it was reasonable to expect in the field, taking into account particularly local conditions such as might be expected in Jordan. Mr Mann understood that the coefficient of variation on the cube test was such that the 3,500-lb.-per-square-inch minimum which had been specified, was in fact quite a reasonable requirement —not too low or too high, but a fair level to strike.

Mr Mann had recently seen the airfield and thought that it was a very fine job. For pilots it offered a good approach for landing and the hills, which it might be felt were too prominent and a possible cause of accident, did not give that feeling, at any rate in daylight.

Mr G. S. Cooper said that when the airfield pavements at Amman had been originally discussed, before the site investigation had been carried out, consideration had been given to the use of soil-cement stabilization, but it had not been possible to proceed with that in view of the great difficulty of obtaining cement. At that time it could not be obtained in the Middle East either from Israel or Egypt, which were the normal sources of supply. In any case it was apparent from the type of soil, with the very high fines content, showing about 80 to 90 per cent passing a 200-mesh sieve, that the use of soil-cement stabilization would have been impracticable.

The soil survey had been done in the early spring, and so presumably the moisture contents would be at their highest. Was any information available on variation of moisture content throughout the year at the construction depth, which was about 2 feet ? Had any experiments been carried out to determine whether the superposition of an impervious surface resulted in an increase in moisture content immediately below the construction ?

For the bituminous surfacing, a good deal of thought had had to be

given to the grade of bitumen to be used, and finally the grades mentioned by the Authors had been selected. The softest grade was 80/100, used in the runway. In practice, had that proved sufficiently hard not to bleed during the very high summer temperatures? A kerosene-resistant compound had been used for filling the joints. Mr Cooper had not yet seen an altogether satisfactory kerosene-resistant joint-filling compound, and more information about that would be welcome. It was stated that the expansion joints in the concrete base slab were spaced at 126 feet 6 inches. Presumably the base slab was exposed to the air for some time before the surface slab was laid. Had contraction joints been provided or had cracks been allowed to form of their own accord? Since the top slab would cover the joints eventually presumably it did not matter a great deal.

Mr Cooper was astonished at the very high flexural strength achieved. It was stated on p. 411 that a 7-day flexural strength of 646 lb. per square inch had been obtained, whilst the 28-day average crushing strength had been 4,750 lb. per square inch. At 28 days Mr Cooper felt that 646 lb. per square inch would give a very high ratio of flexural to compressive strengths, and at 7 days it seemed phenomenal. Could the Authors give any reason for those high figures?

Mr C. E. Andreae referring to the question of rainfall, about which very little had been said in the Paper, asked the Authors for some information on the rainfall at Amman, and particularly the intensity. It seemed that there must be considerable rainfall to justify the elaborate drainage systems, in addition to which it was clear from the Paper that there had been days on which no work could be carried out because of rain. On the other hand, the photographs reproduced in the Paper appeared to show a very arid countryside, which in turn suggested that the rainfall was very slight. Some information on rainfall would, therefore, be of interest.

Mr A. G. Gullan said that since the contract had had to be based on the use of local labour, the labour question was of special interest, but unless some costs were given it was difficult to know whether or not that had been an economic way of carrying out the work. It might be impossible to disclose some of the figures, but if they could be given as unit costs and if some idea could be given of the disposition of the labour force at its peak in relation to the mechanical plant, it would be of value. What had been the overall cost of the construction of the new runway?

It was stated on pp. 395–396 under the heading "Design of Pavements" that the longitudinal gradient was the maximum permissible figure of 1 in 80. That seemed to be somewhat misleading. Mr Gullan understood that the longitudinal gradient of the middle-third of the runway should not vary by more than ± 1 per cent. The maximum resultant "dip" of 1 in 63, referred to on p. 410, seemed excessive. It had been impossible to get a transverse fall of 1 in 100, but Mr Gullan believed that it was not desirable to build a first-class airfield to that standard.

Were the Authors satisfied that the specification adopted for the new runway would meet the needs of future civil air transport using Amman ?

Could the Authors give any indication of the effectiveness of the French drains referred to on p. 401 ? It was a little difficult to understand how those drains could be effective, since the type of material in which they were located was classified as CH A7, which indicated that it was practically impervious but of high compressibility. A grading curve for the material used in the sub-base would be of interest. Mr Gullan noted that the 6-inch-cube tests on the concrete at 28 days had been carried out at Habbaniya. What method had been used to transport the cubes and how it had been arranged to keep the temperature constant ?

Mr D. H. Little asked what loadings the airfield had been designed to carry.

Mr J. A. Dawson, referring to the surface finish of the runways, wondered whether those very fine results had been obtained with the use of placing machines such as the Barber-Greene, or whether all the placing had been done by hand. It had been stated that the concrete finish was very good. Had it been obtained simply from the spreader and finisher, without the use of a belt or other similar method, as frequently employed in the United Kingdom ? One of the slides had shown men working with trowels smoothing the surface. Had the fine finish to the surface of the concrete runways been obtained simply by trowelling the surface after the machine had passed ?

The Authors, in reply, thanked the speakers for their complimentary remarks and dealt first with Mr Snow's question about the effect of elevation of the airfield, the temperature, and the wind direction on the design of the runway. That question had been answered by Group Captain Palmer when he said that to counteract the effects of elevation and temperature the runway had been made as long as possible having regard to economy and safety in the approaches. The orientation of the runway, which had been fixed by topographical features, was 30 degrees off the theoretically ideal orientation obtained from the " wind rose " for the area.

Water, although generally scarce throughout Jordan, was available at Amman. All the water required by the contractor had been pumped from Wadi Ain Ghazal, which flowed approximately $\frac{3}{4}$ mile from the site. Up to 40,000 gallons of water per day had been used for compaction and concrete mixing.

They had not measured the grading of the hand-broken stone, but the principle of the hand-broken stone and clay layers was that the clay provided the fines. They had tested it in wet weather and had been able to roll it after the heaviest rain without any signs of subsidence. As Mr Snow had said, the trenches had been dug without timbering, but the contractor had indeed learned at one time during wet weather, that it was not always possible ; one of the deepest trenches had fallen in, fortunately with no loss of life. Although not legally bound in Jordan to insure his

employees against accident the contractor had in fact insured all his employees. Wage problems had not been the Authors' concern.

As for dimensions on drawings, the Authors had left it to the contractor to make his own conversion from English to metric.

The use of a double slab and not a single slab had been the policy of the Air Ministry when the runway had been designed—if the design slab thickness was more than 12 inches, double-slab construction was adopted because it was considered impossible to compact concrete properly in slabs deeper than 12 inches.

Mr Snow had stated that it had been proved by extensive tests that the type of construction used on the de Havilland Aircraft company's runway at Hatfield was superior to that used at Amman. Mr Snow was probably basing his statement on the experimental tests made at Hatfield,[2] but the Authors did not agree and in fact considered that the opposite was the case. Tests carried out by Air Ministry had proved that when making comparative tests both the moisture and temperature gradients through the slabs should be known. It would appear that at Hatfield full account had not been taken of the effects of the moisture and temperature changes. At Amman they had noticed no loss of strength on account of trimming of the bulge produced by the joint-cutting machine. The cement used was Lebanese cement, and tests had shown that it had a false initial set at 21 minutes and a real initial set at 2 hours 57 minutes. The final set was at 5 hours 31 minutes.

The cave that Mr Bustani had referred to was off the airfield and was an old Turkish petrol store that had been excavated during the 1914/18 war. As a result of the incident the Air Ministry had arranged for a team of geophysicists to check the whole site for caves, man-made or otherwise. The Authors were pleased to say that none had been found elsewhere.

In reply to Mr Mann there had been little variation in soil conditions in the intervening spaces between the trial pits. However, the Air Ministry had extended the runway 500 feet to the west as compared with the original proposals and that had involved an area where the soil type was different. That soil was the Nari referred to on p. 400.

Mr Cooper had asked whether the Authors had noticed any variation in the soil moisture content, the soil survey having been made in the spring, and the runway being constructed mainly during the summer. It so happened that the moisture content remained almost constant at 17 per cent throughout the year, so that even in the summer it was not necessary to add water to assist compaction. As stated on p. 398 the excavated soil had been spread and rolled immediately so as to take full advantage of the natural high moisture content. They had not as yet measured any variation in moisture content under the pavement after construction, but it was to be expected that it would rise slightly. The 80/100 penetration bitumen did not bleed during the summer.

[2] See footnote 1, p. 413.

The joint-sealing compound used was a hot-applied compound containing rubber but no bitumen. It had been developed to meet Air Ministry requirements and was considered to be wholly resistant to the solvent action of kerosene (including Avtag). Contraction joints had not been provided in the bottom slab. A few slabs had cracked during construction owing to contraction. The Authors agreed that the flexural strength of the concrete was high and that had been, without doubt, attributable to the excellent aggregate used. Calcareous aggregates always gave consistently higher flexural strengths than the siliceous aggregates generally used in the United Kingdom; and in addition, crushed aggregates gave higher flexural strengths than rounded aggregates.

In reply to Mr Andreae the rainfall at Amman was 12·5 inches per year and it fell between the end of November and the beginning of March, 3·1 inches having been known to fall in 24 hours. The drainage system was designed to cope with such maximum rainfall. The countryside at Amman was arid during the summer months, when no rain at all fell. During the wet season it was generally green and carpeted with wild flowers, but a few weeks after the end of the wet season it became arid again.

Mr Gullan had asked for details of the unit cost of items. It was not possible to give that information, but the overall cost was approximately £750,000. The labour force at various phases of the contract was given in Fig. 5, Plate 2, and details of the plant in the appropriate sections of the Paper.

It could be stated definitely that machine construction was far cheaper than hand construction for almost all operations, the one possible exception being that it might be cheaper to break stone by hand rather than by crusher plant.

The Authors agreed that the maximum *desirable* longitudinal gradient was 1 in 100, but the maximum *permissible* was 1 in 80. There was no variation in gradient within the middle-third of the runway. Accepting the 1-in-80 gradient and a cross fall of 1 in 100, the resultant " dip " of 1 in 63 was inevitable. The specification should meet the needs of all modern aircraft likely to land at Amman.

The French drains had not been as effective as had been hoped. The soil, although a CH soil, was not like a CH soil in the United Kingdom; it was weathered and granular in appearance and looked more like sand than clay. Water penetrated to a great depth. It also had the effect, however, of choking the French drains in heavy rain by washing the fines into the drain. For a future job the Authors would recommend a cut-off channel.

The concrete cubes had been tested at Habbaniya and transported as priority freight by aircraft. One or two cubes had been slightly damaged, and low results had been obtained from them. No attempt had been made to keep the temperature constant, but the cubes had been boxed in damp sawdust, which had insulated them to a certain extent.

Details regarding the bearing capacity of the airfield, for which Mr Little had asked, were confidential.

In reply to Mr Dawson all paving had been placed by machine except, in the case of the asphalt, narrow bays forming the drainage channels and, in the case of concrete, narrow and curved bays. There had been no attempt to improve on the machine finish of the concrete other than by hand trowelling.

The closing date for Correspondence on the foregoing Paper has now passed without the receipt of any communication.—Sec. I.C.E.

MARITIME AND WATERWAYS ENGINEERING DIVISION MEETING

13 April, 1954

Lieut.-Col. R. H. Edwards, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Maritime Paper No. 26

"Extensions at Takoradi Harbour"

by

Alfred John Clark, B.Sc.(Eng.), M.I.C.E.,

and

John Henry Albert Broughton, B.Sc.(Eng.), A.M.I.C.E.

SYNOPSIS

The Paper describes extensions and improvements carried out at Takoradi Harbour to provide better facilities for the export of sawn and log timber, for the import of general cargo and oil, and for passenger traffic.

For the timber trade, three shallow-water quays, each 500 feet long with adjacent stacking areas, are provided for transit storage and shipping of log timber, and two shallow-water quays, also 500 feet long, equipped with steel-framed sheds for transit storage of sawn timber. The quay walls throughout the timber depot are of in-situ mass concrete founded on a rock bottom.

A new 1,400-foot extension of the existing deep-water quay is described in the Paper and is a gravity wall of concrete blockwork construction. It provides three import berths with a 30-foot depth of water alongside; two of them are equipped with double-storey steel-framed transit sheds. The upper floor of one of the sheds has been fitted out as a passenger landing and embarkation station.

Particulars are given in the Paper of improved oil-discharging facilities, consisting of a tanker berth alongside a properly equipped jetty on the seaward side of the lee breakwater.

Additional railway facilities including reception and despatch sidings, mineral sidings, cargo dump platforms, locomotive sheds, etc., as well as extensions to the road system within the harbour area are also described.

The estimated total cost of the works described is about £4½ million.

Introduction

Takoradi harbour is the only deep-water port in the Gold Coast and is situated in latitude 4° 53′ north and longitude 1° 46′ west. It was built between 1921 and 1928 and is an artificial harbour consisting of rubble breakwaters enclosing a water area of 220 acres. As then completed the internal development comprised deep-water berths alongside for three

ocean-going steamers, berths at buoys within the harbour for eight more, shallow-water quays for lighters and for logs, a coal discharging berth for coastal ships, and a slipway for 500-ton vessels. A small dry dock and a bauxite loading plant and berth were constructed during the 1939–45 War. Moorings under the shelter of the lee breakwater for the discharge of petroleum were provided in 1931.

The port is operated by the Gold Coast Government Railway and was designed primarily as a railway port. Ninety per cent of export cargo and 50 per cent of import cargo are conveyed to and from the port by rail. The use of road transport, especially for the import traffic and for logs, however, is increasing.

Tables 1 and 2 give annual tonnages of various commodities imported and exported during the period 1932 to 1952 and show how the trade of the port has grown since the opening of the harbour.

TABLE 1.—IMPORTS 1932 TO 1952

Year	General: tons	Coal: tons	Oils: tons	Total: tons
1932–33	56,500	21,500	12,000	90,000
1933–34	60,600	28,000	10,800	99,400
1934–35	81,253	34,073	16,483	131,811
1935–36	123,095	42,662	24,359	190,116
1936–37	146,318	53,585	28,318	228,060
1937–38	167,366	76,615	55,231	299,212
1938–39	150,095	68,121	89,193	307,409
1939–40	*	59,663	94,699	
1940–41	*	60,428	125,823	
1941–42	357,960	54,846	114,039	526,845
1942–43	516,681	81,835	117,644	716,160
1943–44	262,952	114,525	96,749	474,226
1944–45	129,089	123,645	84,147	336,881
1945–46	127,920	99,510	72,540	299,970
1946–47	129,522	102,483	80,306	312,311
1947–48	198,181	89,954	98,298	386,433
1948–49	232,509	117,764	128,490	478,763
1949–50	280,857	115,636	130,360	526,853
1950–51	354,688	100,982	167,077	622,747
1951–52	398,608	125,047	167,992	691,647

* Figures not available.

Manganese, which is handled by mechanical loading plant at a single berth alongside, accounts for nearly two-thirds of the whole tonnage of exports, and the existing facilities are considered adequate for present requirements. Bauxite is the only other important export handled alongside, and the remaining export cargoes are loaded into ships moored at the buoys, either from lighters or, in the case of logs, from rafts made up at the timber quays. There has been a rapid expansion of the timber trade and the proportion of sawn timber is increasing as new sawmills are com-

TABLE 2.—EXPORTS 1932 TO 1952

No. of vessels cleared	Year	General: tons	Cocoa: tons	Manganese: tons	Bauxite: tons	Timber: tons	Total: tons
	1932–33	4,479	62,500	73,100		5,726	145,805
	1933–34	2,572	98,595	318,132		5,173	424,472
	1934–35	4,123	79,842	370,525		16,591	471,081
	1935–36	3,609	99,389	470,611		13,528	587,137
	1936–37	5,673	105,024	438,681		16,309	471,081
	1937–38	13,427	42,124	569,203		17,358	587,137
	1938–39	20,723	132 874	272,282		8,848	434,724
	1939–40	*	*	457,357		3,561	
	1940–41	*	*	426,529		26,074	
	1941–42	49,031	89,795	535,827	5,741	33,596	713,900
	1942–43	95,231	76,734	506,343	64,406	35,968	778,682
667	1943–44	114,217	131,491	396,448	144,934	54,505	841,595
566	1944–45	82,743	135,383	644,973	106,352	67,601	1,037,052
618	1945–46	72,597	160,958	709,404	118,772	72,244	1,133,975
654	1946–47	46,488	117,352	826,861	116,390	112,898	1,219,989
734	1947–48	41,954	97,105	624,455	89,506	129,595	982,615
893	1948–49	62,206	128,507	706,060	144,069	175,734	1,216,576
927	1949–50	51,780	164,879	782,055	141,644	151,280	1,291,638
962	1950–51	68,786	165,655	810,265	114,316	232,008	1,391,030
983	1951–52	79,165	128,476	862,575	100,543	223,315	1,394,074

* Figures not available.

nissioned. The tonnage of cocoa exported has remained steady, apart rom seasonal fluctuations, for several years.

Imported general cargoes, mainly machinery, building materials, ehicles, and cotton goods, are handled at the two remaining berths alongide the main wharf; both are equipped with transit sheds, and the asternmost of these (No. 3 berth) serves also as a berth for the passenger nd mail ships which call at the port once a week. The growth in volume f import trade corresponds to the growth of exports and to the general evelopment of the Colony.

Provision had been made in the earlier designs, both for full developent within the harbour shell and for further extensions outside and to the orth of the present harbour. The extensions which form the subject of he Paper represent the full development within the existing harbour. By 937, as the facilities then existing were considered by the Colony insuffient to meet the requirements of an expanding trade, the Government nvited recommendations for extensions from their consulting engineers, Messrs Rendel, Palmer, and Tritton; alternative schemes for partial and r full development were prepared but not executed because of the war. further scheme was prepared in 1947, and formed the basis of the works ere described.

The principal requirements in 1947 were :—

(1) Improved facilities for export of timber, including logs and sawn timber.

(2) Increased berthage for import cargoes and passenger traffic.
(3) Additional sidings to serve the new berths and better arrangements for marshalling and sorting railway traffic.
(4) Roads to serve the newly-developed port areas.
(5) Cranes and services at the new berths.
(6) An oil-import jetty.

Subsequent to the acceptance of the consulting engineers' proposals, a requirement for a new locomotive yard and running shed was added, and during the course of the works further substantial modifications, mainly increases in the facilities to be provided, have been incorporated in the scheme. A general plan of the harbour and a lay-out plan showing original and new works are given in Figs 1, Plate 1.

It was decided that the works should be carried out by contract. Drawings and documents were prepared by the consulting engineers and competitive tenders were obtained, as a result of which a contract for the works was placed in 1949 with Messrs Taylor Woodrow Ltd. The estimated value of the contract was £2¼ million and the contract period 44 months from June 1949. A rise-and-fall clause was included in the contract but owing to increased costs of materials and labour and to additional works subsequently ordered, the final cost will be about £4·52 million, about £0·6 million of the increase resulting from increased costs and £1·67 million from additional works.

Climatic and Local Conditions

Takoradi is the principal terminus and headquarters of the railway. The climate is generally hot and humid and unfavourable to Europeans. Rainfall is of the order of 40 inches per annum and occurs mainly during the period May to October. Although the total rainfall is moderate, the intensity may be high for short periods. The prevailing wind is from the south and during the dry months a land breeze from the north occurs at night.

Sea and littoral conditions are favourable. Heavy seas, as experienced elsewhere, are unknown, although there is an unrestricted " fetch " to the south. There is, however, always a swell which gives rise to surf on the coast and causes slight range within the shell of the harbour. Teredo and martesia are the principal marine pests. Littoral drift is very moderate on this part of the coast and appears to vary in direction with the seasons.

Corrosion is heavy, particularly in those areas directly exposed to salt spray.

The sea-bed off this part of the coast is of rock and consequently the original breakwaters forming the shell of the harbour were extended seawards to obtain a sufficient natural depth at the entrance and to provide deep-water berths at moorings and alongside the main wharf with a minimum of rock dredging. A considerable part of the water enclosure is

therefore shallow and could not economically be deepened, but has been developed for lighterage and log traffic.

Since the coastal hinterland is also of a hilly and rather elevated nature, it is only possible to obtain the requisite land areas for harbour development by a process of levelling and reclamation.

Design and Construction

Timber Depot

The lay-out of the new timber depot is shown in Figs 1, Plate 1. There is a total of 4,100 linear feet of quay in water-depths from 5 feet to 17 feet; of this 1,750 linear feet is used for logs, 1,000 linear feet for sawn timber, and the remainder for harbour craft and lighters.

The purpose of the timber depot is to provide buffer storage at the port for collecting logs and sawn timber delivered by rail and road ready for bulk shipments by vessels at the buoy moorings.

Access by rail is from the west by way of sidings fanning out to the quays and connecting with the main-line system west of the harbour. Road access is obtained from either of the main gates at the north and south sides of the harbour.

Logs are handled at the three northern quays (see *Fig. 2*, facing p. 432) by six 10-ton portal cranes, two on each quay face, and are made up into rafts after being lowered into the water; the rafts are towed by launch to ships at the buoys for loading. Certain species of hardwood which are heavier than water are lightered from quay to ship. Road-borne logs are handled either by crane from the quays or by gravity chute from a site near the root of the main breakwater.

Log storage behind the quays amounts to 60,000 square feet, that is, about 15,000 tons capacity.

Sawn timber is handled through two new sheds in the southern part of the timber depot.

The two sawn-timber sheds are located on each side of "Dock No. 2" which has a minimum water-depth of 7 feet at low water; each consists of ten bays, 40 feet wide and 150 feet long, the bay width corresponding to the length of one railway wagon or half the length of a lighter. Three-ton electric overhead travelling cranes, one in each bay, are used to handle the timber from rail or road vehicles to the shed floor, and thence into lighters in the dock by travelling out over the water on cantilevered trackways.

Timber Depot Quays

The type of construction adopted for the quays was mass-concrete walls cast in situ and founded on the prepared rock of the sea-bed. The filling behind the walls was beach sand, together with a small quantity of shale in areas not subjected to heavy loading. A typical wall-section is shown in Figs 3, Plate 1. The 6-inch nominal projection of the face at datum level

was provided to allow latitude in the placing of shuttering under water, and the lower projections were incorporated to suit the method of shuttering adopted. Except on the quays provided specifically for berthing harbour craft, fixed fendering has not been used, but a bull-nosed renewable 12-inch by-12-inch timber coping has been provided. Bitts, bollards, mooring-rings, and ladders are provided at intervals.

The log-storage areas are surfaced with a 9-inch layer of crushed granite; elsewhere the sand surface has been blinded with a carpet of laterite.

The timber depot section was designed to be constructed first, and with the minimum of disturbance to the existing facilities which were already overtaxed. The contractor elected to work from floating plant and required for access only a few feet of the old 800-foot long timber wharf; his loading depot, stores, and workshops were located on ground fronting on to a section of the south side of the harbour not then required by the harbour authority. The first section of wall built was the north log quay and filling progressively followed the wall construction so that it was possible to have this quay fit for use before the old log quay was demolished. The existing 5-ton log-cranes were transferred as a temporary measure to the new quay and railing of logs was suspended for a fortnight while this was done; otherwise there was no interruption to traffic.

The sea-bed was pitted and excavated where required by rock breaker; final trimming, where necessary to prevent a forward sloping foundation, was done by pneumatic tools.

The floating plant used consisted of four flat-topped 500-ton barges, two of which were shuttering units, and two concreting units. Steel shuttering was used throughout and proved satisfactory, after initial difficulties at tidal levels had been overcome. Each of the concreting barges was laid out for use with bottom-discharging skips for underwater concreting; each had two 21/14 concrete mixers and a mobile crane anchored to the decking.

The contractor first used the " pier and panel " method of concreting the lower portions of the walls, forming sections of wall 10 feet long about 30 feet apart and filling in the gaps as a second operation. About half of the wall was concreted in this manner, and then an " end-on " method was adopted for the remainder. A 300-foot length of wall, built subsequently at the extreme south-west corner of the timber depot as berthage for harbour craft, was constructed using a travelling derrick mounted on the top of the previously built wall and on the fill behind it.

The sand fill was transported by lorry from the estuary of the Whin River, about 2 miles west of the harbour, where it was dug by dragline.

Sawn-Timber Sheds

A section through the timber depot showing the sawn-timber sheds and wharfs is shown in Figs 3, Plate 1. The structures are steel-framed, the

roofs being covered with asbestos-cement " Big-Six " sheets and the walls with galvanized corrugated steel sheets. The columns are set on mass-concrete foundations in the sand fill, and the floors are of 6-inch thick unreinforced-concrete slabs. North lights and ventilators are provided in the roofing. Inside each shed there is one line of railway track back and front and a third line at the rear of each shed for clearing empty wagons. Since there is insufficient room for a shunting neck at the east end of No. 1 shed, a wagon traverser has been provided for the removal of empty wagons on to the service line north of the shed.

The structural steelwork was fabricated in the United Kingdom and, after trial erection of a section in Britain, was shipped to Takoradi. The arrangement was such that it was possible to rivet together the two halves of the columns and to assemble and bolt the girders to the required cambers on the ground, and to thread the girders over the tops of the columns ; this method was in fact adopted. The equipment used for erection consisted of a Bucyrus Erie 54 excavator fitted with a long jib and used as a crane. The steelwork was flame-cleaned or pickled, and painted with one coat of bituminous paint before dispatch. This priming coat, which did not stand up very well to shipment and storage conditions, was cleaned off and replaced where necessary after erection ; the steelwork was then painted with two coats of tropical grade bituminous paint. The wall sheeting was also painted with bituminous paint.

The first concrete was placed in the quay walls of the timber depot in March 1950 and the walls were substantially completed by November 1951, the north log quay being opened to traffic in May 1951. The sawn-timber sheds were occupied by the harbour authority in November 1952 and April 1953 respectively.

The total volume of concrete in the walls is 25,000 cubic yards, the amount of filling about 250,000 cubic yards, and the cost of the quays and sheds, excluding cranes, rail tracks, and roads, was £501,000.

Main Wharf Extensions

The harbour, as constructed in 1928, left scope for development of the main wharf along the lee breakwater for an additional 1,400 linear feet (see *Figs 4* and *5*). In order to make use of this frontage it was necessary to transfer to the north or seaward side of the lee breakwater a berth for loading bauxite which had been constructed during the 1939–45 war.

The main wharf extension provides three short berths or two long ones, the new berths being numbered 4, 5, and 6 with a minimum depth of 30 feet of water at low water. The new berths will all be for general cargo, but No. 4 Berth has special facilities for passengers, and No. 6 berth will generally handle cement imports. At No. 5 berth there is an open dump for cargoes not requiring protection from the weather, and Nos 4 and 6 berths each have double-storied transit sheds. There is a cross-over

between the quayside railway tracks and the rear tracks at No. 5 berth.

The lay-out of the extension is shown in Figs 1, Plate 1, and a typical section of the wharf showing arrangement of cranes, rail tracks, passenger gangway, and transit sheds is given in Fig. 6, Plate 1.

During the war, one bay of transit shed No. 2 was demolished in order to provide for a third railway track and greater quay area. This arrangement having been found satisfactory, it was decided to demolish also one bay of transit shed No. 3 and to extend the third railway track to connect with the quayside tracks on the new berths. A fourth line of rail track has been added on the north side of the sheds as far as the west end of transit shed No. 2 and the breakwater has been widened to accommodate this track and the roadway.

The new main wharf wall is a gravity wall of concrete blockwork in horizontal courses set on a screeded mass-concrete foundation deposited on the prepared rock sea-bed in a similar manner to the timber depot walls. The foundation concrete is 1 : 2 : 4 nominal mix with 25 per cent extra cement, and the blockwork mix is 1 : 3 : 6 nominal, vibrated. The standard blocks used were all 5 feet by 4 feet on the face with depths of 8, 10, and 12 feet to suit the steps in the wall and the coursing. Special taper blocks were used on the rounded corner at the east end of the extension and the face blocks in each course were keyed together with bag-concrete cylindrical dowels or joggles. At each end where the wall joins the breakwater or the rubble bank of the old wharf, the rubble was excavated and prepared in steps corresponding to the block-coursing, and mass concrete-filling sections were cast in situ. The top of the blockwork is at + 1·0 foot and coping level is + 10·0 feet. Between these levels the wall is formed in situ in mass concrete and incorporates a service subway. Timber fendering is provided on the face of the wall. The bulk of the filling was shale obtained from Cox's Fort hill on another part of the site and, since the physical properties of this material are variable and somewhat unreliable, a backing of granite rubble was tipped behind the block wall, between it and the fill.

The cranage provided consists of four 3-ton portal cranes to each berth, straddling the quayside railway track. Since these cranes travel on a continuous track, more than four can be used at a single berth by robbing one of the adjacent berths. It was decided late during the construction period that an existing 15-ton crane being fitted with a portal undercarriage should also travel on the main wharf crane tracks.

Construction of Main Wharf Wall

The construction of the main wharf extension was commenced at the end of 1950, the contractor having established a blockcasting and stacking yard at the east end of the main wharf and on a part of the site occupied

by No. 3 transit shed which was partially removed to make sufficient room.

The method adopted for block-laying was to employ a specially designed 15-ton electric crane which travelled along the top of the blockwork on trestles, building ahead of itself, up to a level of — 4 feet; at this level the temporary crane trestles were placed ahead and the crane travelled forward. The final course of blocks between + 1·0 foot and — 4 feet was laid behind the crane after removal of the trestling. The western return end and a short commencing length of main wall on which the travelling crane was erected, were built using the 15-ton derrick in the contractor's block-yard. To facilitate block-laying on the return end of the wall, the block-setting crane was fitted with bogies capable of being rotated through 90 degrees. When the main length of straight wall had been completed and a portion of the curved section built, the crane was travelled forward on to temporary steel trestling until it had reached the alignment of the return end; the bogies were then rotated so that the crane travelled forward on the new alignment. Since the heaviest blocks could not be lifted safely in air at extreme radius they were lowered into the water with the jib at short radius and luffed out to position under water. In addition to safe-load indicators, a positive-acting overload-prevention cut-out was fitted to this crane.

The concrete-screeded foundation was built ahead of the blockwork using the floating concreting plant from the timber depot. Deposition of filling was kept at a sufficient distance behind the scarp end to avoid interference with the divers and to suit the general excavation programme.

The mass-concrete capping was placed from floating plant, steel shutters being used.

The method of construction is illustrated in *Fig. 7* and Fig. 8, Plate 1.

The block wall was substantially completed in January 1953, 37,500 cubic yards of mass-concrete foundation and block-walling having been placed in 21 months.

Transit Sheds Construction

Because the sheds were built on more than 40 feet of newly deposited shale filling, piled foundations were required. These piles were cast at a yard just south of Butua Lagoon, transported by rail, and driven to rock by a single-acting steam hammer working in light leaders fitted to the jib of a Bucyrus Erie 54 excavator. The number of piles driven was 660, and the average rate of driving was four per day. Construction of pile-caps and ground-beams followed the driving and the Bucyrus Erie 54 excavator served as a crane for the erection of the steelwork.

The two sheds are both steel-framed buildings with concrete-block walls and " Big-Six " asbestos-cement sheeted roofs. The column foundations are groups of 14-inch-by-14-inch reinforced-concrete piles driven

through the fill to rock at a batter of 1 in 12. Ground floors, being on new filling liable to settlement, consist of a layer of hand-packed granite pitching stone with a surfacing of bituminous macadam. The first floor of No. 4 shed is in filler-joist construction, designed for a superimposed loading of 2 cwt per square foot, and of No. 6 shed in reinforced concrete for 3 cwt per square foot loading. Some of the heavy section beams are welded flanged beams. Corrosion protection is similar to that adopted in the sawn-timber sheds.

A plan of transit shed No. 4 showing the arrangements made for passenger traffic is given in Fig. 9, Plate 2 ; the upper floor of this shed is devoted to passengers, high-duty goods, and a suite of offices. Communication between the upper floor and the ground floor and cargo platforms is by electric lifts of 3-ton and 5-ton capacity in Nos 4 and 6 sheds respectively.

Bauxite Berth and Ropeway

The mining and export of bauxite in the Gold Coast was commenced in 1942. The ore is obtained from Awaso, 127 miles north of Takoradi, and transported by rail to a dump north of the harbour. Here it is loaded as required and conveyed by an aerial ropeway to the bauxite berth where it is discharged by conveyor loaders into the ships' holds. The original berth was constructed inside the harbour for security reasons ; it consisted of four mass-concrete piers supporting the steelwork carrying the transporter conveyors, the shore end of the buttress girders being founded on the rubble of the lee breakwater. The ropeway was routed over Cox's Fort hill and along the crest of the lee breakwater and the alignment contained three changes in direction.

The harbour extension works necessitated the following :—

(1) The removal of the bauxite berth to another site to permit the extension of the main wharf.
(2) The re-routing of the ropeway so that Cox's Fort hill could be removed.
(3) The re-modelling of the railway sidings feeding the ore dump to suit the new lay-out of the railway yard.

Since a total shut-down of the berth for a period was unavoidable, the British Aluminium Company arranged to accumulate working stocks of bauxite at the port of Burntisland and the change-over was planned to reduce to the minimum the time of total shut-down ; care was taken to phase the other operations to avoid interference with the transfer of the bauxite handling facilities. A section through the berth showing the loading facilities is given in Fig. 10, Plate 2, and *Fig. 11* shows the bauxite loaders in their new positions.

Opportunity was taken to investigate and rectify depletion of the seaward rubble slope of the lee breakwater at the ropeway and berth founda-

Fig. 2

View of Timber Depot showing Log Quays and Sawn-Timber Sheds

Fig. 4

MAIN WHARF WALL, SEEN FROM ENTRANCE CHANNEL

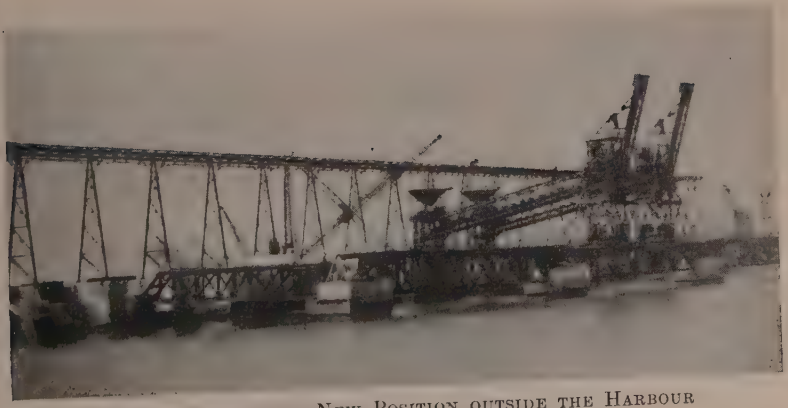
Fig. 5

MAIN WHARF WALL, LOOKING EAST FROM EXISTING WHARF

Fig. 7

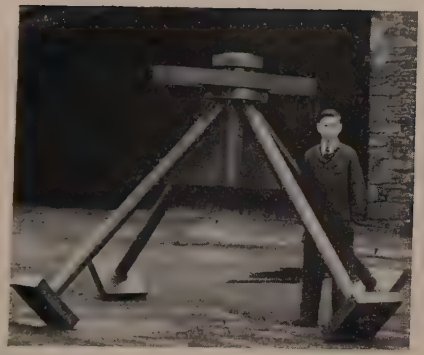
Extension of Main Wharf. Block-Setting Crane at Work

Fig. 11



Bauxite Loaders in New Position outside the Harbour

Fig. 12



HOLD-OFF MOORINGS. ANCHOR FOR EMBEDMENT IN ROCK BOTTOM

Fig. 14

VIEW FROM NORTH OF HARBOUR SHOWING NEW RECLAMATION

tions, and to underpin the foundations where necessary. The section shows the profile of the breakwater before and after strengthening.

The underwater seaward slope of the breakwater was generally found to be steeper than originally designed, and some loss of stone had occurred above low-water level from the action of swell, which had also caused the displacement of some of the bagged concrete work on which the mass-concrete piers were built. Defective bag-work was replaced by mass concrete cast in situ and, at the berth, the breakwater slopes were made up to a section rather greater than the original section. Precast concrete blocks, each weighing about 1 ton with the corners chamfered, were used for this work since no plant was available at the quarry to handle large granite blocks. The precast blocks were placed partly by rolling down the slopes, partly by lowering on tackle, and partly by dumping overside from a barge. About 10 per cent of granite rubble was added.

The new berth was sited outside the harbour alongside, and under the shelter of, the lee breakwater nearly opposite its original position. The foundations of the ropeway steelwork were modified as necessary to accommodate the shore ends of the four buttress girders and the steelwork carrying the shore ends of the conveyors. The steelwork, which had been carefully maintained, was found to be in good condition and was re-used, the only modifications being that in the new berth the level of the buttress girders was raised to give easier access for painting, and the main girders were placed farther apart over the new piers so that some desired additional length of berth was obtained.

In order to give greater security in northerly storms, hold-off moorings anchored (*Fig. 12*) by concrete plugs into the rock of the sea-bed have been provided so that vessels can tie up to these moorings from the bow and quarter, and lie a few feet off the berth during loading. The timber fendering at the piers, installed to take small accidental bumps, is carried on steel frames supported in sockets at sea-bed level and is lashed by chains to the tops of the piers. Bolsters of saplings or bamboo are inserted between the fender frames and the piers to give a cushioning effect. Consideration is being given to the replacement of the bolsters by flexible rubber units in order to reduce maintenance.

Apart from minor modifications required to cause the buckets to tip before reaching the end wheel, no changes were required in the ropeway on the lee-breakwater section. From the breakwater to the ore dump, the new route over the reclamation is in a straight line clear of the sidings. Existing pylons and angle-station steelwork were modified as necessary and re-used. The foundations have been taken down to rock, the angle station foundation being a rubble mound supporting the concrete bases, and the remaining foundations being plain concrete piers. Where the ropeway crosses the open beach small concrete caissons were sunk through the sand and hearted with mass concrete.

No particular difficulty was encountered in the ropeway re-alignment,

and the main limitations on the construction of the berth were imposed by the necessity for keeping the old berth in operation during construction of the foundations for the new one and by the limited working space on the breakwater arm. Two steam derricks were used for handling the lighter steelwork and for foundation construction and were moved as circumstances required. The main girders and buttress girders were floated round, one at a time, on barges which were fitted with ballasting pumps to assist the falling tide when the girders were lowered on to their bearings.

The hold-off mooring anchorages were excavated by pneumatic tools, the divers working inside a shutter which held back the mud, and were concreted from floating plant. The anchorage cables were also laid from floating plant.

Oil Berth and Pipelines

The oils imported are white oils, Diesel oils, and fuel oils. Bunkers are supplied at certain berths. For 20 years tankers used to tie up stern-on to the lee breakwater and lie-to two anchors. Three pipelines leading to the tank installations, 2 miles north of the harbour were connected with the tanker valves by lengths of flexible piping secured to pontoons. At the new berth, also at the north side of the lee breakwater, but not yet completed, the tankers will lie beam-on to the berth. It is of the breasting block type. Hold-off moorings, similar to those at the bauxite berth with which they are connected, are provided and rubber blocks have been used in the fendering system to absorb kinetic energy during berthing. The steelwork of the berth is designed as a horizontal portal frame to resist lateral forces from the ship's springs.

The general arrangement and a section of the oil berth are shown in Figs 13, Plate 2. The depth of water alongside will be 33 feet, but the pier foundations have been taken down below the sea-bed (rock) to a level of 38 feet below L.W.O.S.T. to allow for possible future deepening of the berth. The deck lay-out includes hose-handling derricks and water services. The steelwork for this berth is protected by coal-tar enamel applied hot.

Re-alignment of the oil pipelines was required throughout their length in order to clear sites for new roads and railway tracks. The re-alignment was done by the oil companies in three main stages, one pipeline at a time, the other two remaining in use. The stages were :—

(1) Along the lee breakwater the pipes were moved to the north side of the bauxite ropeway.

(2) In the railway yard area and northward to Butua Lagoon the pipelines were re-routed across the reclaimed area and thence along the foreshore to Butua Lagoon.

(3) From Butua Lagoon to the north end of the locomotive yard the pipelines were diverted across the new culverts and clear of the turn-table area.

Some ingenuity was required to fit the various moves in with the onstruction work.

Bunker lines with connexions for diesel and fuel oil are provided in the ilway at the main wharf extension.

Railway Yard

The land at the back of the harbour is hilly and the road and rail pproaches to the port were constructed in cuttings. The restricted pproach formed a bottle-neck which would have prevented the exeditious handling of additional traffic for which the harbour extension orks were designed. Between the railway cutting and the sea, Cox's ort hill, an eminence about 80 feet high and containing about $1\frac{3}{4}$ million ubic yards of material, stood in the way of further developments ; it was onsidered necessary to remove it either wholly or in part. The hill consted of a core of shale which had been found in previous works to be hemically active when exposed to air, owing to the presence of carbon and ulphur, overlaid by a cap of inert laterite. The chemical activity of the hale had been found in laboratory tests to be arrested when the material as deposited under water, and on the works where the shale was used for ll, it was generally deposited below high-water level and blanketed with a hick layer of laterite. It was known that there would be a surplus of aterial if the whole hill was removed and provision was made in the conact for the surplus to be dumped at sea ; at the same time the reclamaon-area north of the harbour was considered not fixed in extent, and the eclamation was in fact increased in size to absorb most of the material hich would otherwise have been surplus. The reclamation was, therefore, ade as large as was considered safe with little knowledge of the amount f bulking which would occur as a result of excavation. Since the amount f inert fill from the excavation was insufficient to blanket an area which ould absorb all the surplus shale as well as to provide the filling at the mber depot and the top layer of fill at the main wharf extension, it was ecided after consideration of costs to proceed with the enlarged reclamaon and to make up the deficiency of inert fill by bringing in beach sand om the nearby Whin River for the timber depot instead of laterite from e hill. Some economy was effected in that piles were not required for e sawn-timber shed foundations, and the sand provided a perfectly onsolidated fill.

The seaward boundary of the reclamation (*Fig. 14*) was protected from our by a rubble dyke of trapezoidal section.

The whole new area of flat land made available in the railway yard ea as a result of excavation and reclamation was about 95 acres.

The lay-out of the trackwork in the railway yard may be divided into :—

(1) Mineral sidings.
(2) General cargo sidings.
(3) Marshalling and sorting sidings.

The mineral sidings provide five tracks 1,500 feet long for manganese trains, on which there is continuous traffic, the ore being usually delivered by tipper direct from the railway wagons to the conveyor loading plant. Sidings are also provided to a manganese dump and a coal dump on the reclamation, and two bauxite sidings 1,000 feet long with a connexion through the aluminium company's tipper to gravity-fed empty wagon storage sidings.

The cargo sidings serve two cargo platforms 950 and 1,100 feet long by 100 feet wide, a 3,000-ton capacity cement shed, and minor storage buildings and dumps. Reception and dispatch sidings are located between the cargo platforms and the main-line tracks, and shunting necks are provided at each end. A carriage and wagon repair shed has been built at the south end of the railway yard.

The handling of cargo on the platforms is designed to be by portal cranes transferred from the timber depot, two of which span the tracks between the platforms, supplemented by mobile cranes. Lighting is being installed to permit 24-hour per day operation.

Work on excavation of the hill was commenced in June 1950 and proceeded at an average rate of about 40,000 cubic yards per month until late in 1953. The digging equipment used consisted of two 54 Bucyrus Erie face-shovel excavators for about half of the period, and one for the remainder. It was necessary to loosen the shale by blasting before digging and 3 or 4 auger drills were constantly employed. The distribution of the spoil was by 10-cubic yard Diesel-driven Euclid tipping wagons assisted by bulldozers at the dumping grounds.

The rubble dyke was built by end-tipping from Decauville wagons and the working face of the dyke was kept a little ahead of the filling throughout. No serious loss of fill by wave action was observed during the works. As the seaward face of the dyke assumed its natural slope under the action of the sea, large boulders and pieces of concrete from demolitions were tipped over the face for additional strengthening.

The trackwork was specified to be to Gold Coast Railway standards. The gauge is 3 feet 6 inches and rails are 80-lb.-per-yard flat-bottom section dogspiked to creosoted timber sleepers. Unfortunately, owing to failure of the railway creosoting plant, it was not possible to pressure-creosote the sleepers but it was decided to hand-apply the creosote rather than leave them entirely unprotected. The ballast used was specified to be $\frac{1}{2}$-inch to $1\frac{1}{2}$-inch crushed granite 10 inches thick under the sleepers. Turnouts are generally 1 in $8\frac{1}{2}$ and minimum curve radii 400 feet.

The railway work had to be arranged so that disturbance to port operation was reduced to the minimum.

Road access to the railway yard is by a new road (the Northern Approach Road) running parallel to, and west of, the main-line tracks, which conforms to the town planning requirements for the development of the district. The crossing of the main line is to be by a skew bridge approached by a viaduct on the south side and by an embankment from the north. In view of difficulty in obtaining steelwork and the advantages of prestressed concrete in erection and resisting corrosion and attack from fumes, this form of construction has been adopted for the bridge spans. The general arrangement of the bridge and viaduct is shown in Figs 15, Plate 2.

The two bridge spans consist of precast beams 44 feet 6 inches long, stressed by the Lee McCall system with in-situ concrete filling between and above them.

The approach viaduct is in reinforced concrete and consists of continuous 25-foot spans with expansion joints at 125-foot centres, and a simply supported end-span.

The bridge foundations are in mass concrete because the sandstone is near the surface at that point. Under the viaduct the rock levels are lower and piled foundations are, therefore, provided for the viaduct columns.

The roadworks generally consist of 30-foot single carriageways on reservations and are, where practicable, 50 feet wide. The foundation consists of 10 inches of hand-packed granite pitching with a bituminous macadam base course $2\frac{1}{2}$ inches thick, a wearing course $\frac{1}{2}$ inch thick and a sealing coat of sand and bitumen brushed in. The mixes are in accordance with B.S. 162. For the surfacing work the aggregate was not heat-dried but no surfacing was done in the rainy weather.

Locomotive Yard

The original locomotive yard for the harbour was sited at the north end of Cox's Fort hill cutting and consisted of a two-bay running shed with turn-table, coaling, ashing, and watering points. Owing to the inadequacy of these facilities for the increased traffic in and out of Takoradi harbour and the need for replanning and extending the harbour sidings, it was decided to build a new and larger locomotive depot on another site. The choice of sites near to Takoradi was very limited by the hilly nature of the country ; the most convenient and economical site was considered to be in the vicinity of the Butua Lagoon outlet. Since there was insufficient length of site south of the Lagoon outlet, it was necessary to locate the turn-table and northern approach turn-outs on the north side of the Lagoon.

The connexion between the running-shed and turn-table area involved the culverting of the Lagoon east of the main line, and a similar and shorter culvert was constructed west of the main line to carry a road diversion.

As a result of investigations made at the site, it was decided to design for a flow of 1,200 cusecs ; the upstream culvert was founded on rock and the downstream culvert on a reinforced-concrete raft. The arrangement of the culverts is shown in Figs 16, Plate 2.

The new running-shed is a four-bay steel structure 225 feet long equipped with inspection pits, water points, ash pits, wheel-drop, workshops, and stores. The smoke-troughing and roofing are of asbestos-cement sheeting. A coal dump and ash disposal points are provided in the lay-out, Figs 1, Plate 1, and a typical section of the shed is shown in Fig. 17, Plate 2.

The site now occupied by the running-shed was used as an open dump for general cargo prior to the extensions and could not be released until the new cargo platforms were ready for service. Construction of the locomotive yard was commenced in September 1952.

The culverting of the Lagoon was carried out within earthen and block-work dams in conjunction with a temporary stream diversion and the programme of work was designed so that construction should be completed during the dry-weather flow, with provision for flooding the dams if the flow should become dangerous.. This work was successfully completed.

Cranes

The new cranes provided are for operation on A.C. 3-phase, 50 cycles, 400 volts, and are as follows :—

Sawn-timber sheds

Twenty electric overhead travelling cranes (Joseph Booth and Brothers)

Rating	3 tons
Hoisting speed	30 feet per minute
Travelling speed	250 „ „ „
Traversing speed	80 „ „ „
Track centres	37 feet 6 inches

Timber depot log-cranes

Six electric portal cranes, level luffing (Stothert and Pitt)

Rating	10 tons
Hoisting speed	60 feet per minute
Slewing	1½ revolutions per minute
Luffing speed	120 feet per minute
Travelling speed	45 „ „ „
Gauge	16 feet
Maximum radius	40 „

General cargo cranes

Twenty electric portal cranes (including four for cocoa wharf, and four for existing wharf) (Stothert and Pitt)

Rating	3 tons
Hoisting speed	3 tons at 200 feet per minute, 2 tons at 300 feet per minute
Slewing	1½ revolutions per minute
Luffing speed	150 feet per minute
Travelling speed	55 „ „ „
Gauge	18 feet 1½ inches
Maximum radius	60 feet

The 10-ton and 3-ton portal cranes have fluid couplings incorporated n the slewing motion. Their jibs were zinc-sprayed during manufacture s an additional protection against corrosion, since these cranes will perate in situations much exposed to salt spray.

The 10-ton portal-crane clearances were designed to be clear of the ailway loading gauge but are less than the normal structure gauge, whilst he 3-ton portal cranes were designed to clear the structure gauge.

In addition to the foregoing, an existing fixed 15-ton wharf crane is eing equipped with a portal-type undercarriage to 18-foot-1½-inch-gauge nd the existing logging cranes, D.C. operated, are being transferred to he railway yard to work on the cargo platforms.

Electricity Supply

The electricity supply is provided by the Government Electricity epartment at 6,600 volts, 3 phase, 50 cycles, and is transformed down 400/230 volts for general power and lighting distribution within the arbour and railway area.

Five substations were found necessary and the location and estimated adings of these are as follows :—

Substation	Area served	Power : kVA	Lighting : kVA	Total : kVA
A	Timber depot	525	100	625
B	Main wharf	517	88	605
C	Marine	94	14	108
D	Railway yard	305	175	480
E	Loco. yard	61	28	89
		Estimated loading :		1,907

Whilst the accepted policy is to adopt an A.C. supply throughout the rbour and railway area, six of the existing portal cranes equipped with

D.C. motors will be serviceable for some years and to supply the necessary D.C. power, duplicate rectifier equipments of 250 kW output each, have been provided in the railway yard sub-station.

The ten overhead travelling cranes in each timber shed are supplied with power through duplicate underground feeders and changeover switches connected to a single overhead bus-bar system of distribution running throughout the length of the sheds; this system also supplies power to a wagon traverser and a capstan, and facilitates the connexion of any additional equipment which may be required in the future.

Duplicate underground power feeders are also provided to supply the portal cranes; these feed alternate plug boxes seated at ground level and at approximately 50-foot spacing throughout the length of the crane tracks. Each plug box has fuse protection.

Underground cables have, in many cases, to be laid solid owing to the presence of sulphur shale.

For the illumination of the railway yard and log quays "floodlighting" has been adopted, using ninety 1,000/1,500-watt projectors, mounted on 70-foot-high galvanized steel towers. This provides an average lighting intensity of 0·7–1 lumen per square foot over the whole area.

The switching of these floodlights is effected by a series of contactors remotely controlled by push buttons situated in the traffic operating staff office, about 800 yards distant.

The switching of all lighting is arranged so that when work is not in progress, the intensity can be reduced to that suitable for police patrol of the area.

For road-lighting, sodium vapour lamps mounted on 25-foot-high reinforced-concrete columns, to give Class "A" standard of lighting, have been installed.

Railway lighting at points and crossings has been provided throughout the various areas.

Quarry

The whole of the stone used in the works was quarried from the railway quarry at Sekondi, which was handed over to the contractor for the construction period. The quarry had supplied all the stone for the original harbour construction and was still in use in 1949, though on a very small scale, for production of railway ballast by hand knapping.

The quarry face at the commencement of the works was about 800 feet long and varied in height from 40 feet to 70 feet. The over-burden of laterite about 30 feet thick, was near to the rock face, and had to be cleared back before operations could commence. It was excavated by hand and tipped in a natural valley some distance behind the face. Additional removal of over-burden was done intermittently during the works as required.

The rock is granite. Veins of platy biotite, recognizable by its softness and crystalline nature occurred occasionally, and with any decomposed rock at the top of the granite, were excluded from the work. This granite had stood up well to sea action in the breakwaters, no weathering being visible at old drill holes in the stone. The density is 174 lb. per cubic foot.

The method of quarrying was to blast to give a preponderance of stone capable of being handled by two or three men, and to " pop-hole " the larger rocks down to these sizes. The man-size rock was then transported by Decauville wagons either to the crushers or direct to railway wagons as pitching stone. Quarry waste was used for blinding road foundations. The crushing plant was arranged to provide railway ballast (2 inches to 1 inch nominal) or concrete aggregate of 3-inch or 1-inch maximum size as required.

The total production of the quarry from 1949 until October 1953 was 131,500 cubic yards of pitching stone and 172,000 cubic yards of crushed stone. The quantity of explosives used was about 0·4 lb. per cubic yard of stone and the quantity of over-burden removed was 100,000 cubic yards. There is a direct rail link to the harbour which lies about 7 miles to the west.

Sand

The littoral in the neighbourhood of Takoradi consists of surf-washed sand beaches, with a number of rocky outcrops, mainly of sandstone, but on the east side of Sekondi there are a number of granite outcrops. Most of the sand used locally for building is taken from the beach either at Whin River or at Sekondi, but this sand is calcareous, consisting mainly of ground-up shell and, being flaky, is not very suitable for use in high-grade concretes. Near the granite outcrops, east of Sekondi, the sand is composed mainly of quartz grains with a low proportion of shell ; this sand was used for underwater and reinforced concrete. The supply was insufficient for all concretes and various mixtures of the quartz and shelly sands were used depending on the properties desired in the concrete.

Cement

Cement was imported in 1-cwt paper bags and, provided it was not stored too long, these bags seemed effective in keeping moisture out. During the busiest part of the work, all cement arriving at the port was carried from the quarantine anchorage by surf boat ; a proportion of this cement was lost as a result of wetting.

Timber

Various local timbers were used. Kussia was used in all situations exposed to attack by marine organisms ; kussia and dahoma for railway

sleepers; mahogany, sapele, and iroko for building work; wawa, and local plywood for shuttering. All fendering timbers were pressure-creosoted.

Acknowledgements

The Authors desire to record their appreciation to the General Manager and Harbour Authority, Gold Coast Railway, for permission to publish the Paper and to Messrs H. F. Merrington, B.Sc.(Eng.), M.I.C.E. and H. R. Boyce, B.Sc.(Eng.), A.M.I.C.E. of the Consulting Engineers' and Resident Engineer's staff respectively for reading over the drafts and for assistance in preparing the drawings.

The main contractor was Messrs Taylor Woodrow (West Africa) Limited. The agent until 1952 was Mr J. Guthrie, who was subsequently appointed Gold Coast General Manager of the Company; he was succeeded by Mr T. Fairclough. A list of the principal sub-contractors is given in the Appendix.

The Paper is accompanied by twenty-two photographs and twelve sheets of drawings, from some of which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared, and by the following Appendix.

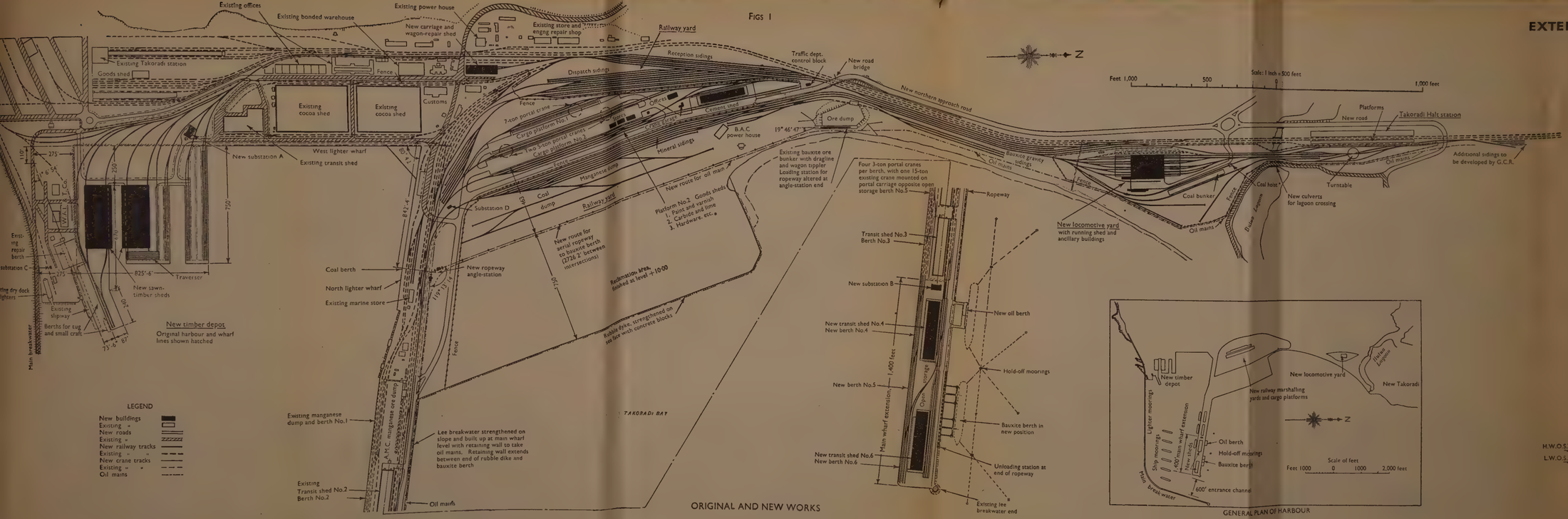
APPENDIX

Contractor and Principal Sub-Contractors

Contractor	Messrs Taylor Woodrow (West Africa) Limited.
Sub-contractors :—	
Bauxite ropeway and steelwork	British Ropeway and Engineering Co. Ltd.
3-ton electric overhead travelling cranes	Messrs Clyde Cranes and Joseph Booth and Bros.
3-ton and 10-ton portal-cranes	Messrs Stothert and Pitt Limited.
Bituminous paints	Messrs B. Bowran Ltd. Wailes Dove Bitumastic Ltd.
Lifts	Express Lift Co.
Electrical installation	British Insulated Callender's Construction Co. Ltd.
15-ton crane modifications	Sir William Arrol Ltd.
Asphalt	Faldo Asphalte Co. Limited.

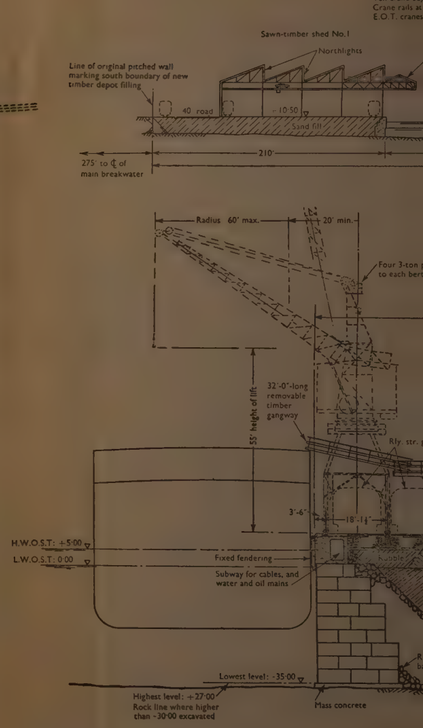
EXTENSIONS AT TAKORADI HARBOUR

FIGS 1



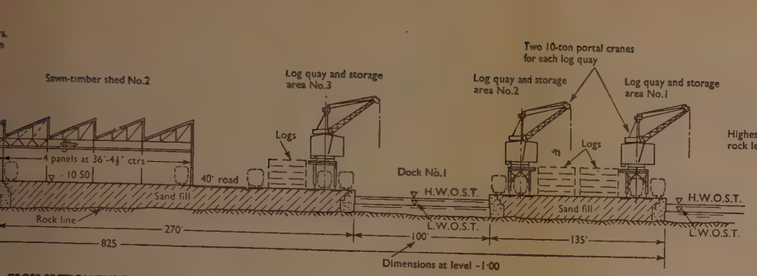
ORIGINAL AND NEW WORKS

GENERAL PLAN OF HARBOUR



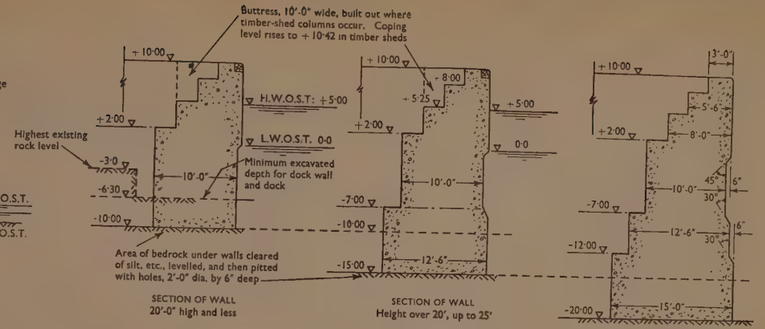
& Sons, Limited : London

FIGS 3



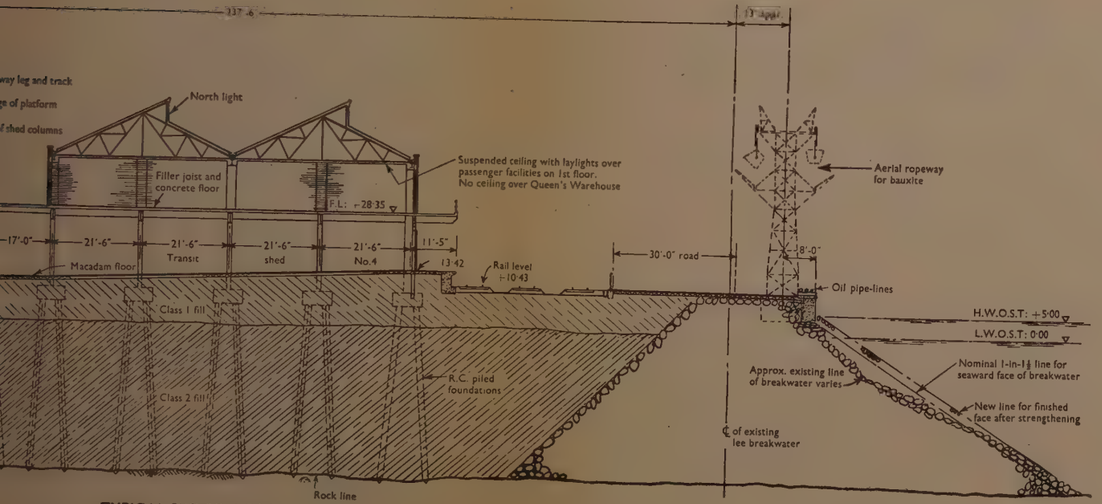
CROSS-SECTION THROUGH DEPOT
Scale: 1 inch = 100 feet

TIMBER DEPOT



TYPICAL SECTIONS OF MASS CONCRETE WALLS
Scale: 1 inch = 16 feet

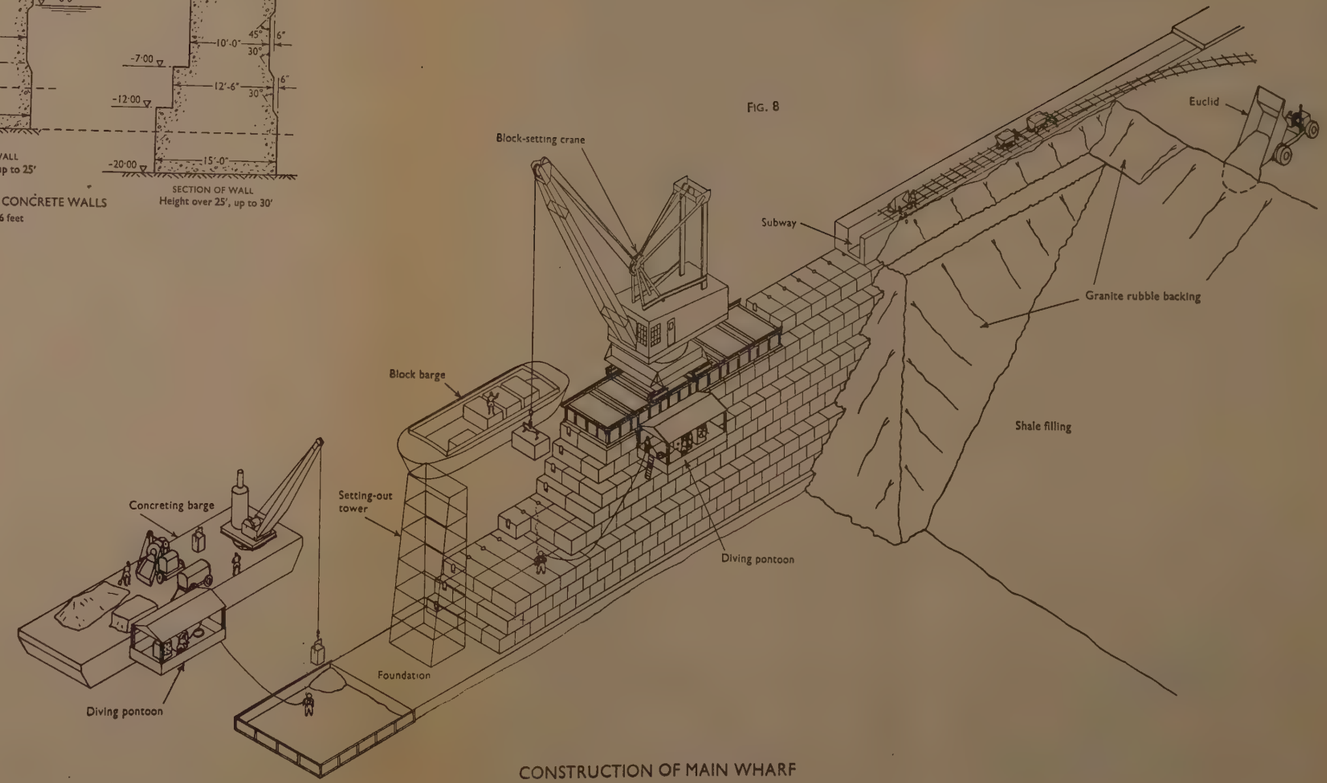
FIG. 6



TYPICAL SECTION OF MAIN WHARF

Scale: 1 inch = 32 feet

FIG. 8



CONSTRUCTION OF MAIN WHARF

Discussion

Mr J. H. Jellett referred to the statement made on p. 428 that, in the imber-depot quays, no fixed fendering had been provided; apart from eference to the 12-inch-by-12-inch timber coping beam, no other mention ad been made of fendering. Was some form of floating dummy rovided ?

How was the bituminous paint on the steelwork for the sawn timber heds behaving ? Experience had shown that the behaviour of that type f paint could be very patchy and a record of how it had lasted in the onditions in Takoradi would be of interest.

It was unusual to berth ships in what was, after all, the open sea when pecial unloading or loading plant (such as that at the bauxite berth) had o be provided. Were sea conditions ever experienced at those berths hich made it impossible to operate the loading plant ? In the descrip- on of the construction of the berths, there was reference to the fact that he face of the breakwater had been drawn down to some extent by sea ction, which indicated that conditions of calm did not always obtain in hat part of the harbour.

Turning to the reclamation, which was described on p. 435, Mr Jellett ked if the cost of blanketing the shale filling over the whole of the clamation was considered to be justified. So far as could be seen from he Paper, no immediate purpose was envisaged for a large part of the clamation area, and even though it had been made as large as possible, me of the surplus material had had to be taken to sea. In those circum- ances, would it not have been more economical to allow the shale over e unwanted part of the reclamation area to disintegrate, which pre- mably would have made room for that extra volume of material which d to be taken to sea ? It might be that there was something undesirable allowing such a large volume of shale to disintegrate in the vicinity of e harbour, but the point had not been brought out and it was interesting.

Finally, information on some of the timbers mentioned in the end of e Paper would be of great value. Nowadays people were offered a rge variety of timbers from various parts of the world to take the place some of the better-known qualities and species which were not so entiful as in the past, and information about the behaviour of the new mbers was not always readily forthcoming.

Mr A. J. Hill said that he could comment on certain matters from e contractor's point of view, particularly on the background in West rica which accompanied the work and the many important circum- nces which had had to be taken into account before the work was dertaken.

The job was tendered for in 1949, which was in what one might scribe as the early part of the post-war period when many things were

uncertain. It was in a country where there had not been a vast amount of civil engineering practice in previous years, mostly because of the war, and where there was a developing political situation—a situation which, during the course of the work, saw a change in the form of government. There was also the fact that the community at Takoradi was of such a size that the work would have a great impact upon it. There were about 600 Europeans at Takoradi and about 15,000 Africans. The advent of harbour extensions, therefore, meant a 10-per-cent increase in the number of people there, so that not only was there the impingement of the work on the harbour itself but there was also a question of its effect locally in social and other types of contact.

Lastly, there was the circumstance of the great difficulty with materials, especially those which had to be imported into the country. All those factors had been reviewed and some deliberate policy decisions had been taken so as to get the best out of the work.

Those policy decisions included, first of all, a determination to utilize Africans to the utmost on the work in all its phases so as to minimize the number of Europeans who had to be sent from England. That had involved training, teaching, and encouraging Africans to do work of which previously they had had little or no experience.

To accompany that decision, it had been decided to move very much in the direction of the mechanization of operations. The mechanization which had been embarked upon was, he believed, far ahead of what had been expected by people who had seen previous work in the Colony.

Considerable attention had been given to the personnel side of the work, both with Europeans and Africans, and some specific arrangements had been made about the housing, welfare, and health of the people. Lastly, it had been decided that in view of the shortage of materials and that that was one of the first post-war major overseas contracts, at least on a firm price, service from the United Kingdom would have to be faultless and that extra trouble would have to be taken to see that the people on the job were served.

The average African labour force on the job had been about 1,600 men, rising to a peak of a little less than 2,000. During the course of the job, the wage of the African had radically changed. The labourer's wage had risen from 2*s*. 9*d*. a day to 4*s*. 6*d*. a day—a 64-per-cent increase—and the wage of a typical operator, doing a skilled job, had risen from 5*s*. a day to 9*s*. a day. The introduction of mechanization had been a great buffer against the impingement of extra labour cost on the work. Among the Africans there had been the usual number of tradesmen such as carpenters, masons, riggers—but a lot was done—and it needed a good deal of determination and patience on the part of the Europeans concerned —to train and encourage men to do other skilled jobs. There had been twenty electricians and welders on the job, most of whom had been brought on during the course of the work. Eventually, fifteen men,

entirely without previous experience, had become divers. Forty fitters and other men had been employed on the servicing of plant and, in all, there had been about a hundred plant operators. Mr Hill did not claim that the quality of the labour so trained had been equal to that obtainable in Great Britain, but a lot had been done to help the Africans to play a bigger part in the work.

The average number of Europeans on the job had been forty-two, rising to a peak of forty-nine, of whom fifteen had been employed on management and supervisory work, eleven on technical work, eleven on supervising mechanical servicing, seven as divers, and five on clerical duties. During the course of the work, twenty men had had the company of their wives and there had been eight children living at Takoradi.

During the work, eight pupil engineers had been sent (in pairs) to Takoradi as part of their indentured training under the scheme between the Institution and the Federation. Their tours had lasted about 9 months ; they had all thoroughly enjoyed it and most had been reluctant to return home to make way for others.

In the mechanization, every effort had been made to use African operators on the powered units, and at no time were Europeans regularly driving any machine. The Europeans instructed the Africans, and after that the Africans drove the machines. There were 120 powered units on the job, ranging from the largest crane to smaller items weighing $\frac{1}{2}$ cwt. Some benefit had been derived from the background of the mining industry in West Africa, and from that it had been possible to build a greater ability in the Africans to work machines.

One point in the Paper reflected rather unfavourably on the excavation. On p. 436, the average rate of digging was stated as 40,000 cubic yards per month. That included any time which those machines spent on crane work, pile-driving, and so on. Mr Hill's records showed that normally, when digging was at its peak, the average rate was 100,000 cubic yards per month. Altogether, 80 per cent of the time had been spent in working, 6 per cent in mechanical breakdowns, and 14 per cent in standing due to various causes, weather being one. The machines had achieved a very creditable output, with an average throughout the digging work of 70 cubic yards per hour per cubic yard of bucket capacity—a good performance in Britain or anywhere else.

The measure of the degree of mechanization was one powered unit for every 150 Africans ; but Mr Hill said that if the job had had to be done again mechanization would have been developed even further.

There were one or two points on the subject of welfare which might be of interest to those who had been to, or contemplated going to, similar places. First of all, there existed a local hospital, with facilities both for Europeans and for Africans, and nothing special had been provided in the contract. The Europeans voluntarily had 3-monthly examinations to clear their bills of health and to make them feel confident and happy—

an essential for good work. On the contractors' staff there had been altogether fifty-one hospital cases, which worked out at one case per 44 man-months on the job (which meant that on the average each person went to hospital once during the currency of the contract). The average duration of hospitalization was about 7 days, but that was more precautionary than anything else and there had been no serious illness. There had been malaria cases, skin disorders, and intestinal troubles, but they had all been of fairly short duration.

There had been no fatalities among Europeans on the work, but unfortunately one of the resident engineer's staff had died from poliomyelitis at Takoradi. Two others of the staff (a man and his son) had also contracted the same disease and had been sent home.

The contractors had had very little to do with medical care or housing of the Africans; the housing of the Takoradi community under the Government had been arranged independently of the contract.

Two fatalities had occurred among the Africans, which was a ratio of one fatality to every 45,000 man-months on the site.

Mr Hill said that there had been a great deal of trouble in getting plant and materials from the United Kingdom in the prevailing conditions of universal shortage. Weekly schedules had been prepared in the fullest detail for all equipment and materials being shipped to Takoradi so that the people on the site had known exactly where everything was and there had been no doubt about what was happening.

One of the great difficulties had been the time taken to get something from England to West Africa. The average time had been from 10 to 13 weeks. The transit time, by boat, to West Africa was 3 or, at most, 4 weeks, and the remainder of the time was occupied in delays in the United Kingdom in getting a boat—for boats had been generally very full at that time— or in incredible delays at the other end. Only those with experience of the frustrations of the quay-side in Africa would recognize what he meant.

Those delays had produced one benefit: they had made people look ahead, and that was important to any industry.

Mr H. R. Boyce said that it should be borne in mind that Takoradi was not a terminal port but a port of call. Very few ships discharged more than 600–2,000 tons. Ships were coming and going the whole time, proceeding to other ports on the West Coast of Africa, and the only goods which were handled in bulk at Takoradi were manganese and bauxite (ships on charter entered light and departed fully laden); the shipment of bauxite was intermittent because the demand was also met from other parts of the world. The only cargo imported in bulk was coal, which was required for the Gold Coast railways, who were almost the sole importers of coal. Some Diesel train sets were on order.

On p. 430 mention was made of the extra width obtained during the war of the quay or wharf at berth No. 2. At that time there had been

rge shipments of crated aircraft arriving from America, ultimately to be own to Cairo and other parts of the Middle East. Additional space had een required. With more cars and lorries being shipped into the country, hat space had proved very valuable. The small cased goods had gone traight into the transit sheds. Cars had been off-loaded and parked any- here on the roads in the neighbourhood of the transit sheds, causing ongestion.

A glance at berth No. 5 (Figs 1, Plate 1) would show that there was a pace between sheds Nos 4 and 6. In view of the bulky goods which ame into the harbour, Mr Boyce felt that that was a very useful space. t was not big enough for a full berth, but a ship coming in with a lot of ulky goods would be able to berth there and discharge the goods on to he large paved area between those two sheds.

The major buildings were mentioned in the Paper, but there were also number of small buildings—substations, small stores for inflammable oods such as carbide, and some small workshops. The walls of those mall buildings were of blockwork. The substations had concrete roofs; he small stores and workshops had local timber-made trusses. The russes had metal connectors, such as were used in Great Britain.

Referring to the railway yard, Mr Boyce said that at one time it was o have been sited rather inland from its present position, and the old narshalling yard (to the north of the locomotive shed), on the extreme ight of Figs 1, Plate 1, was to have been retained in use. After the orks had started, the Railway Administration had given further thought o the matter and had decided to have the marshalling yard brought in s close as possible to the harbour. That meant that the lay-out had ad to be fitted behind the cargo platforms shown in Figs 1, Plate 1. The ay-out had certain disadvantages, but the Railway Administration had onsidered that having all the shunting locomotives and operations within he harbour area was a great advantage which outweighed the disadvan- age of the cramped working.

Commenting on the underwater concrete, he said that the aggregate as of crushed granite, which was not so good as a rounded aggregate. he concreting equipment for the underwater work had all been operated rom floating craft—large barges equipped with two mixers and swing eigh-batchers. The batchers had been a constant source of trouble to he contractor. They had seemed frail, and as soon as a labourer threw shovel full of stone into the batcher the whole set-up had shuddered; was not long before the batchers had been thrown out of adjustment. requent repairs had been necessary.

Two methods of concreting had been used in the timber depot. At rst it had been done by constructing a number of piers and then con- reting from pier to pier—the pier-and-panel method. Later, when the ater became deeper, the " end-on " method had been used. Mr Boyce elieved that the pier-and-panel method had the greater advantages.

There was greater flexibility for the contractor and the general work was better than in the deeper sections where the " end-on " method had been used. Underwater concrete was deposited and screeded to form the foundation for the main blockwork wall, and it was also used in the piers for the bauxite and oil berths. The concrete, generally, had a good finish and the amount of poor work was small. The placing and spotting of the skips had to be watched and individual spotting had been essential.

Mr P. A. Scott had made several visits to Takoradi in another connexion and, through the courtesy of the engineers, he had recently been able to see round the works described in the Paper. He thought it remarkable that the increase on the original estimate of £2¼ million was only £½ million in view of the fantastic way in which costs had risen in the Gold Coast during the past 5 years. The contract was on a bill of quantities—it was not a " cost-plus " contract—and he wondered whether that was why the cost was so close to the estimate.

Anybody who had seen the works would have received two particular impressions. The first was the magnificence of the provisions made for the timber—covered sheds for sawn timber and fine handling appliances. The port authorities, who had authorized that provision, had been extremely generous to the timber considering the treatment which the original timber merchants or the forest owners gave their own timber and the treatment which the timber received when it reached Europe. Neither at source nor at destination was it given the care which it received at Takoradi harbour.

The second impression gained was the very fine workmanship in the passenger shed, shown in Fig. 9, Plate 2. The entire 400-foot length of one upper storey of one transit shed was given over to passenger accommodation, which must be equal to the best in Great Britain, with the possible exception of that at Southampton. It was certainly of a very high standard, and Mr Scott was curious about the fact that those sheds would be used for only a short period—about 3 hours a week—when the mail boat arrived. It had not been arranged that they should have other uses during the remainder of the period. In other ports in countries which were perhaps not so wealthy, the passenger had his luggage examined and his passport checked in a shed which could be converted, later, into a cargo shed.

Turning to Fig. 6, Plate 1, he said the inner crane rail seemed to present a foundation problem, because the outer crane rail was resting on the sea-wall. Piles could not be driven into the rubble bed ; the normal fill above the rubble was class 2 or class 1, which apparently would accept piles, but apparently piles could not be driven there either, so it looked as though the rubble backing to the wall had had to be extended upwards to support the rail beam. Was he correct in that assumption ?

The anchor for the hold-off moorings (*Fig. 12*), described on p. 433, was

nteresting. The Authors had not said very much about the way in vhich that interesting mooring had been sunk into the rock and concreted n. It looked as though they had had to make holes, to set the moorings n the holes, and to concrete them in. Was that so ?

He endorsed the views on mechanization expressed by Mr Hill. The ime of the shilling-a-day African labourer had disappeared long ago. Even in those times, it was not cheap labour. With present prices for abour, mechanization was as important in Africa as in any other part of he world. The work done at Takoradi had undoubtedly been a great enefit to the Gold Coast, because the country had a very big develop- nent programme in hand and many of the men trained to handle machines t Takoradi were at present being used in other parts of the country to rive heavy machines in the Gold Coast development programme. In all hat they were thinking and planning in the Gold Coast today, it was enerally agreed that all heavy civil engineering, to be economical, must e based on the fullest possible mechanization. That was from the finan- ial point of view as well as because there was a certain labour shortage.

Mr G. A. Wilson approved the provision of such excellent passenger ccommodation but he certainly did not agree with the suggestion that nything would do for the present-day shipping passenger. Airfields had et a high standard. For what volume of passenger traffic had the ccommodation at Takoradi been designed ? There were two grades— eck passengers and cabin passengers.

He had seen some figures recently which suggested that fast Far Eastern passenger liners had increased their draught by 4 feet in the past 0 years. The draught of cargo vessels was said to have been static at bout 30 feet for about 30 years, but it was known that there was a endency for the displacement of cargo vessels to increase from 8,000 to 0,000 tons, and also for their speed to increase. The Suez Canal was 34 eet, but at the annual general meeting of the Company it had been nnounced that dredging was to be done to pass vessels of 36 feet draught. or some years it had been forecast that beam was the most likely dimen- on to increase but, from the preceding figures, there now appeared to be tendency to increase draught, which presented a more difficult problem.

From the Paper, it would seem that the draught of berths Nos 4, 5, and ranged from 27 feet through 30 feet to what looked like 35 feet. Probably squat " did not matter very much when a ship was moving slowly into arbour ; allowance had to be made, however, for pitching or rolling,and nce there seemed to be a certain amount of swell in the harbour, perhaps feet would be needed. A further 2 feet might be allowed for list, so that r the average cargo ship with a draught of 30 feet, it could be argued that 5 feet of water would not be excessive. It seemed that berths Nos 4, 5, nd 6 had not quite enough water to provide for the future ; on the other and, if the harbour was to be further extended, the deeper berths could e made in the future.

Similar remarks regarding draught applied to the oil berths. The depths ranged from 33 feet to 38 feet, whereas he understood that the Americans were considering 40-foot depths. The berth was on the seaward side of the arm, where there seemed likely to be a certain amount of movement requiring some clearance under the ships. The ore carrier was the most difficult craft of all, for there was a tendency to build those ships with mechanical unloading devices, which would lead to bigger ships.

He was interested in Mr Boyce's remarks about the middle section in the lay-out of the quays—1,400 feet with two sheds and an open space. He did not like the idea of half-berth ; if it were occupied, a ship could not be left in either of the others, and if it were not occupied it was wasted. He thought it was for practical purposes a two-berth lay-out with a space in between, and in that case the argument for a little more depth was strengthened.

The passenger accommodation was in No. 4 shed, and if in future a service of fast passenger liners was anticipated, that berth might require the greater draught at an earlier date than the others.

Mr Scott had suggested that timber received better treatment at Takoradi than elsewhere. Mr Wilson felt that possibly he had not recently seen the timber storage at the London Surrey Commercial Docks.

Mr D. H. Little referred to the north lights in the roof trusses. They actually faced north, but he assumed that that did not matter in that part of the world—they had simply to be vertical. If that were so, he would have thought that in Fig. 6, Plate 1, the light was coming from the north and that the northlight roof would throw light only on to part of the shed which already had windows and not into the middle of the shed.

In any case, why were north lights needed ? Was it to keep the heat of the sun out or to keep the daylight out ? The roof of the building was not very well insulated—corrugated iron, painted black. If it was necessary to keep the sun out, had the Authors considered using ply-glass with an ordinary pitched roof, because the northlight roof was 20-per-cent dearer than the pitched-roof type of construction ?

Mr Little next mentioned the settlement of filling. From Fig. 6, Plate 1, it appeared that there was nearly 45 feet of filling, and he thought that would settle as much as 2 feet in 30 years. Presumably the floor of the transit sheds would have to be made up to suit in the course of time. Reference had been made to the crane track partly on solid wall and partly on filling. What happened when the rear leg settled ? He had seen bigger crane tracks laid on sleepers on filling, with the intention of packing the sleepers up as settlement took place, but he did not see how they could readily be packed up in the case in point. Had there been much evidence of settlement yet ?

He assumed that the shale was what was known in Devon as " shillet." The whole of the Devonport dockyard had been filled in that way, about 1900, and by 1937 there had generally been at least 2 feet settlement.

One vexed question concerned the block walls. All walls built by British engineers in the tropics seemed to be mass gravity walls. They might have been all right hitherto, but previous speakers had already mentioned that labour costs were getting higher and higher in the tropics. That sort of wall had gone out of favour in Great Britain; the last ones built were the big monolith walls at Southampton and they had proved very expensive. All British consulting engineers used such walls in the tropics, but were they the right answer and how much longer was that use to be continued?

It had been made clear that the sea-bed at Takoradi was rock, but what type of rock was it—granite or shale? Steel piles could be driven into shale and even into a stiff chalk bed, and it seemed that the embankment with the piled platform going over it, as had been used on the bauxite wharves, might be the type of wharf which engineers would soon be compelled to use. If they must hold the earth up—although why they deliberately set out to hold the earth up he did not know—he thought the continental development of the steel sheet-piling, also used in Britain, was a cheaper alternative. That question of design in Britain for the tropics was very important, and he thought the time would come when gravity walls would become less popular. The maintenance people were probably delighted to have gravity walls, but in Britain they cost at least three times as much as the sheet-pile wall, and even in Takoradi, if they cost £400 or £500 a linear foot, an open wharf could be provided for half that money.

Those who had quoted costs had declared that the days of the 1*s.*-a-day labourer were a thing of the past. From that arose the most important issue in the Paper.

Mr B. F. Saurin said that a good many years ago, when all oil berths had been very flimsy affairs, hold-off moorings had usually been provided to prevent the full weight of the ship bearing against whatever staging or pontoon was provided for the oil-pipes. Nowadays substantial alongside berths were usually provided. A good example was given in the Paper. It would be of interest to know whether the hold-off moorings were a traditional survival from the older system or whether there was some special circumstance in the sea conditions which made them desirable.

He noted with interest that the fendering provided at the two ends of the oil berth took the form of rubber blocks in compression. Could the Authors state what energy capacity was provided in each of those assemblies of rubber blocks?

There had been mention of the use of hot-applied coal-tar enamel. Perhaps when considering the other question regarding the success or otherwise of the bituminous material, the Authors could say how the coal-tar enamel, applied hot, was enduring.

On the subject of corrosion the Authors had said that the waters were very aggressive and corrosive. Was concrete subject to corrosion in those

parts, and had that been a factor in the selection of the type of main wall ? So far as Mr Little's remarks were concerned, Mr Saurin agreed that perhaps the blockwork wall was indeed a little unusual for a modern piece of work. The answer might be that reinforced concrete would be too liable to attack from sea water.

Mr Little had suggested the alternative of the sheet-pile wall in steel piling. He would himself suggest that if, instead of being an English job, it had been an Italian or French job, it would probably have been done by building reinforced-concrete caissons in the harbour, floating them to the site, sinking them, and filling them up afterwards with whatever material best suited the circumstances.

The Authors, in reply, said there was no fixed fendering in the timber-depot quays except the hardwood coping which had been mentioned. Three of the quays were used only for putting logs overside into the water and the timber coping was to prevent damage to the quay-edge when handling the logs. In the water the logs were their own fenders so to speak. At the quays of the sawn-timber sheds there might be some reason for fendering, but the water was reasonably calm, the finish of the concrete very smooth, and the lighters which worked in the dock were provided with timber rubbing strakes. Conditions for the lighters were better than when lying alongside ships and so far there had been no complaints.

Regarding reclamation and filling, no filling had been taken to sea ; it had all been deposited in the reclamation. It was difficult to say what extra expense had been involved in separating the fill into two classes instead of using it as it came from the hill, for there was an all-in rate for disposing of it in the manner described. The decision to blanket the shale fill had been based upon experience obtained during the original harbour construction ; there was a great demand for level sites near the harbour and the blanketing would enable the reclamation area to be brought into use earlier than would otherwise have been possible. The so-called first-class fill had not been altogether first class. It looked like laterite but in fact contained a high proportion of clay. The only really first-class fill on the job was the sand used in the timber depot, which, being deposited into water, was excellently consolidated.

Bituminous paint on the steelwork had proved rather disappointing. The steelwork had been flame-cleaned and given one shop coat of bituminous paint but during shipment and while lying at Takoradi the coating had started to fail. The steelwork, therefore, had become badly rusted and all of it had had to be cleaned off and reprimed. It was too early to say what lasting protection would be derived from the finished bituminous painting, but in the timber depot sheds the steelwork was protected from exposure by the sheeted walls.

Mr Jellett had asked about the timbers mentioned at the end of the Paper. Kussia had been chosen for underwater fendering timber mainly as a result of the Sea Action Committee report on tests carried out at

Takoradi, and appeared to be standing up very well. Local experience had shown the other timbers to be suitable for the purposes for which they had been used, although mahogany, sapele, and iroko would be considered too expensive for ordinary building work in the United Kingdom. Wawa (obeche), which was light in weight, inexpensive, and easily worked, was sold in the United Kingdom for common joinery work and mouldings.

Mr Scott had thought that rather too good a job had been made of looking after the sawn timber. In Britain, probably the sides of the timber sheds would not have been sheeted, but a good deal of salt-water spray came over off the reef and timber exporters had pressed for complete protection from weather. Most of the sawn timber was quite valuable furniture wood, which shipped at about 10*s*. a cubic foot.

The timber exporters had given an assurance, as had the railways, that the wood would be brought down in closed wagons to the port and the handling methods had been designed on that assumption, but in fact most of it had arrived in open trucks.

The crane-rail beam, to which both Mr Scott and Mr Little had referred, had been a rather troublesome problem. Originally, in the belief that the first-class filling would be laterite gravel, it had been proposed to put the beam on a timber grillage of longitudinal and cross-sleepers ; it had been reckoned that it would be packed up when it settled. The material, however, had proved more clayey than anticipated, and so under the crane rail it had had to be dug out and replaced with rubble stone, and quarry waste, down to the level of the top of the rubble backing of the wall. Then the weight on the rubble filling had been spread by a wide reinforced-concrete beam. There was almost certain to be some settlement, and at sometime or other the rail would have to be raised and packed up.

The passenger landing station in No. 4 transit shed occupied 256 feet of the total length of the upper floor and provided accommodation for 250 deck passengers and 150 cabin passengers. Whether the passenger accommodation was really necessary was a matter for the port authority. Probably Mr Jellett had a lot to answer for. When people saw what had been provided at Southampton they expected something of the sort in Takoradi. Rather similar accommodation had recently been provided at Sierra Leone.

Regarding depths of water available at the various shipping berths, it should be remembered that the original harbour had been constructed on a rock bottom with the breakwaters going out far enough to obtain 38 feet at the entrance. The depth aimed at for the new berths on the main wharf was a minimum of 30 feet and some rock dredging was being done departmentally to obtain that depth at No. 4 Berth.

The latest Elder Dempster mail steamers drew 25½ feet, the permissible draught being at present controlled by available depths at Lagos, but there would be no difficulty, apart from expense, in deepening the berths at Takoradi. The bauxite berth had a depth of 32 feet (rock level was about

35 feet) but so far the vessels chartered for that trade had not exceeded 26¼ feet draught. At the oil berth 33 feet was provided to suit tankers of 12,000 tons drawing 29 feet. The oil companies would have liked to bring in larger tankers of greater draught but the amount of additional rock excavation in the berth and the approaches would have made it very expensive.

Mr Wilson had used the expression " half-berth," but in the Paper, the Authors had described the 1,400-foot extension of the main wharf as providing three short berths or two long ones. The open space on the quayside between the two transit sheds was occupied by the railway connexion considered to be necessary from the front of the end berth to the lines at the back of the sheds. That end berth would be used for bulk import of cement and it would cause congestion to have that amount of railway traffic passing along the front of the other cargo wharfs.

Replying to questions regarding the northlight roofs of the timber sheds, the Authors considered that so long as the building was north of the Equator the vertical glass should be put on the north side. That had the effect of keeping out most of the direct rays of the sun whilst letting in the light from the sky. Also, the vertical glazing faced away from the weather and would be much less likely to give trouble than glazed lights in a pitched roof. If pitched roofs had been adopted the Authors would probably have used corrugated Perspex sheets as in the locomotive shed.

In the original harbour all the quay walls, both the shallow ones and the deep water quays, were made of reinforced concrete, in the form of a beam-and-slab deck construction carried on cylinders. About 5 years after they had been finished, quite a lot of defects had developed, and they had all had to be gunited. Corrosion was very active in tropical sea exposures and the most trouble-free type of wall, when foundation conditions were good, was a gravity wall of masonry.

The cost had not been so very much in excess of a steel wall or a reinforced-concrete wall. After all, it was right in principle, to make use of local materials, and for a concrete blockwork wall, the only imported material was the cement. A steel sheet-pile wall with sulphur-shale filling at the back of it and the tie rods buried in that material would have been a very doubtful proposition, apart from the difficulty of constructing it before or while the filling was put in.

Mr Saurin had asked about the hold-off moorings at the oil berth. Although the oil berth was outside the harbour—the same remarks applied to the bauxite berth—it was on the lee side of the lee breakwater and therefore generally fairly calm. Occasionally there would be a choppy sea when the " tornadoes " blew from the north off the land but except for a few days in the year there was no great trouble. The oil companies had asked for only 3½ feet extra draught to be allowed below the bottom of the ship, on account of scend, etc. The hold-off moorings added to the safety of the oil and bauxite berths when strong winds blew from the north,

ut they had not been asked for by the oil companies. The British Aluminium Company asked for them at the bauxite berth, where the structure was not designed to carry bollards, and then the Marine Department of the Harbour Authority had asked for them at the oil berth also, although that jetty was very substantial and was equipped with bollards.

The special anchors for the hold-off moorings had been concreted into plug-shaped holes cut in the rock bottom by divers using pneumatic tools. The holes had been slightly undercut to give a better grip for the concrete.

In reply to Mr Saurin's question about the energy capacity of the oil-berth fenders, each fender was provided with eight Goodyear rubber blocks having a total energy capacity of 530 inch-tons for 10 inches compression.

It was too early yet to say how the coal-tar enamel, to which Mr Saurin had referred, was standing up.

The closing date for Correspondence on the foregoing Paper has now passed without the receipt of any communication.—Sec. I.C.E.

RAILWAY ENGINEERING DIVISION MEETING

11 May, 1954

Mr W. K. Wallace, Vice-President, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Railway Paper No. 53

"The Wear of Steel Rails : a Review of the Factors involved"

by

John Dearden, M.Sc., F.I.M., A.M.I.Mech.E.

SYNOPSIS

A new instrument has been used to measure the wear of rails of different compositions and of rails situated in widely varying conditions of traffic and atmosphere. On straight or slightly curved track which is typical of most lines on British Railways, atmospheric corrosion has been found to have such a dominant effect that it usually masks any difference in the rate of wear of different compositions in the same track. Sorbitic rails were more wear-resistant than normal rails of the same composition, but suffered from corrugation and shelling. No difference could be detected between acid-Bessemer and basic-open-hearth-steel rails of the same nominal composition, nor between high-carbon- and medium-maganese-steel rails.

Rails have been found to wear more rapidly in industrial than in rural atmospheres for the same traffic. The lowest wear per million axles was found in the dry clean atmosphere of the London tubes.

Annual wear is considered to be directly proportional, not to the annual traffic, but to the square root of the number of axles passing per annum, because of the influence of corrosion.

The profile and condition of the running surface has been closely examined and observations made which may help to explain the process of wear.

Introduction

The annual purchase by British Railways of about 220,000 tons of steel rails, costing about £6,500,000 at 1953 prices, to replace rails worn in service, indicates the scope for economies which could be achieved by the introduction of a more wear-resistant rail. If this quantity of rails be assumed to renew 1,400 miles of track and sidings, the old rails of which have lost an average of $\frac{3}{4}$ inch from the running surface, then the metal lost through wear amounts to 45,000 tons, or only 20 per cent of the total weight of the rails when new. Because of this small loss of section, the remaining 80 per cent has to be sold as scrap at a value which is only 40 to

15 per cent of the current price for new rails. If the cost of the new rails less the credit from the scrap rails) be charged to the $\frac{3}{4}$ inch of metal removed by wear, this portion of the rail is found to have cost approximately £90 per ton; or more strictly, the wear of rails provokes a net expenditure on new rails of £90 for each ton of worn metal, exclusive of the cost of laying in the track.

It is not surprising, therefore, that engineers, steelmakers, and metallurgists, continue to seek more wear-resistant rails having a prolonged life, so as to delay their inevitable periodic renewal at a high cost.

Previous Papers[1–7] presented before the Institution have recorded the development of the rail from pre-1860 wrought iron, through high-carbon steel, silicon steel, and sorbitized steel, to post-1932 medium-manganese steel. The authors of these Papers have usually stressed the importance of safety, so that the pursuit of wear resistance has necessarily been tempered by the need to have rails free from defects and tough enough to resist the severe impacts they have to endure in service.

A review of these Papers gives the impression that railway engineers have been reluctant to state anything more than their opinions on different grades of rail steel, and the very few numerical values of rail wear which can be found in the literature have been expressed in so many different terms that their comparison is difficult, if not impossible. It is not surprising that Law and Harbord[7] complained of the lack of information from which conclusions could be drawn regarding the respective merits of different types of rails, and they pointed out that engineers frequently held opposite views on the same class of rail.

During the discussion of Dalby's Paper,[6] Ellson mentioned the very wide variation in wear which he had found in ordinary steel rails situated in twenty different places and subjected to similar (but not exactly the same) traffic conditions. When wide and inexplicable variations occur, there is an obvious reluctance to publish results and an inconsistency among the few available values of rail wear.

It is evident that more factors are involved than have so far been measured or even taken into consideration, and the main object of the Author's research was to recognize these factors and assess them if possible. His engineering colleagues in the London, Midland and Scottish Railway's Research Department had previously (1937 to 1939) attempted to study the relative wear of different combinations of rail and tire steels by means of a small-scale rail-and-tire machine, but this had proved impracticable because of the slow rate of wear and the corrugation of the rolling surfaces. Laboratory wear tests have the advantage that wear can be accurately measured under controlled conditions, but when they do not reproduce service conditions it is difficult to translate their results into practice. The rail-and-tire machine operated in a laboratory atmosphere and was thus

[1] The references are given on p. 480.

protected from the influence of outdoor atmospheric corrosion, which was then suspected of accelerating the wear of rails and tires.

It was, therefore, decided to transfer attention to the wear of rails in the track, and measurements were at first confined to straight track at locations where trains were neither accelerating nor braking, so as to eliminate a few of the many factors involved in rail wear. Loss of section caused by battering and pounding was also excluded from the investigation.

Measurement of Rail Wear

A variety of instruments were available, ranging from calipers to the more complicated instruments which record the profile of the entire rail section. Calipers have the advantage that a large number of measurements can be made easily and quickly at different places along the same rail, but they measure only loss of depth and not loss of section.

The latter can be recorded by one of the many profiling instruments available, but which may be inconvenient to use on lines carrying very frequent traffic.

All these methods suffer from the lack of a permanent datum surface to which the worn profile can be referred when determining the amount of wear, and they require the elapse of a considerable number of years before the amount of wear can be assessed even with moderate accuracy. Another method consists in weighing the entire rail periodically, but this requires possession of the track, and the result is affected by the corrosion of the non-running surfaces ; this can cause a gain in weight during the early stages of corrosion, and a subsequent loss in weight when the accumulated rust is shed or scraped off the rail.

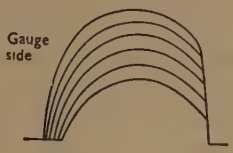
Although these methods may be satisfactory for the measurement of long-term wear in connexion with track maintenance, a more precise method was considered necessary for research purposes. The problem was therefore referred to Mr J. Shaw, of Healey, near Rochdale, who designed and constructed the Rail Contorograph illustrated in *Fig. 1*, facing p. 464.

The contorograph records the profile of the running surface only, with a 10 : 1 vertical enlargement but no horizontal enlargement, thereby making it possible to measure wear in one-tenth of the time required by same-size methods. It can be clamped to the rail in about 5 seconds and a quick-release device permits its removal in about 2 seconds. The non-running surfaces of the rail to which it is clamped form the datum for measurement and are protected from corrosion by grease during the intervals between measurements, which are normally made once a year at the same position on the rail. *Fig. 2* shows a typical set of profiles made at yearly intervals.

The vertical magnification of the profile and the radii at the corners of the rail head limit the recorded profile to about $2\frac{5}{8}$ inches of the $2\frac{3}{4}$-inch width of the rail. The area between successive profiles can be measured by planimeter and divided by 10 to give the actual loss of section from the

unning surface ; but in most cases it is simpler and sufficient to measure he loss in depth at the apex of the rail profile, and most of the results vhich follow are expressed in this way. Normally, eight profiles are re-orded and averaged for each type of rail involved, and a period of ½ to hour is needed for two persons to record the eight profiles and to clean nd grease the clamping surfaces on the rails.

Fig. 2



The contorograph was originally designed to be clamped to 95-lb. bull-ead rail, but by means of suitably shaped packing pieces it can also be sed on the various flat-bottom rails now in use.

An inspection of more than three hundred measurements has shown hat 70 per cent are within 10 per cent of their appropriate mean value for measuring period of 1 year. When the period is increased to 2 years, he proportion within 10 per cent of the mean rises to 85 per cent. Standard eviations from the mean of four measurements on a single rail are generally to 4 per cent for one year's wear, and 1·5 to 3·5 per cent for 2 years' wear. ontinuing the measurements for 7 years, or increasing the measuring ositions to twenty per rail does not appreciably reduce the standard eviation below these limits.

Measurements are normally made annually, and continued for at least years in order to check the result obtained during the first year. Some f the earlier projects were continued for longer periods to compare short-rm with long-term wear. The right-hand (or 6-foot) rail has been con-dered separately from the left-hand at situations on the former L.M.S.R. ystem after it was discovered that the wear on the right-hand was averag-g 10 per cent more in depth than on the left-hand on steam-operated lines. his was ascribed to the increased corrosion of the running surface arising ut of its repeated wetting by the continuous blow-down of locomotive oiler water then in operation. No difference in average wear between ft- and right-hand rails has been observed on lines of the former L.N.E.R. ystem or on electrified lines.

Some Features of Rail Wear

) *Profiles of Worn Rails*

The exaggerated profile recorded by the contorograph reveals features rail wear which are not so obvious in same-size profiles. Starting with a

new rail, wear is more rapid at the flange corner than elsewhere. If the radius of the crown is 12 inches there is also rapid wear on the outer corner, and several months elapse before there is appreciable wear at the centre of the rail ; but with the more recent 9-inch radius, rapid wear at the outer corner in the early stages of service has been much reduced.

On a busy line carrying about ninety trains per day wear during the first 3 months proceeds at about twice the subsequent rate, and at least 6 months' running-in is required to remove about 0·02 inch before the wear settles down to a steady rate. This preliminary rapid rate of wear is thought to be caused by the decarburized layer of steel forming the skin of the rail, and by the change in profile of the running surface.

During the first year or two the running surface takes on a profile which then remains practically unchanged for at least the next 8 years, except in the circumstances described below. This profile moves downwards with vertical wear and also horizontally if lateral wear occurs. It is common to most rails on straight and slightly curved track of correct gauge, and is marked A in *Fig. 3*. This represents the profile found on steam-operated lines of the London Midland and the Eastern Regions of British Railways. Profiles on some electric lines are slightly flatter.

It has not yet been possible to follow the profile of the same rail for longer than 9 years, but an examination of many worn rails from various lines (including curves) has shown that some departure from this common profile does ultimately occur in certain circumstances. The departure is confined to the outer half of the running surface and it occurs at places where there is abnormal lateral wear. When the majority of the wheels run with their flanges pressing hard against the rail the contorograph profile tends towards that marked B in *Fig. 3*. This condition is usually matched on the opposite rail by profiles tending toward C in *Fig. 3*, especially if the track is wide to gauge or the tire flanges have worn thin and the treads hollow. This relation of tires to rails is illustrated in *Fig. 4*. Under these conditions the wear on rail C has been found to be as much as twice that on rail B. It has been observed that the shape of the profile on the gauge half of the running surface is the same in all these cases.

If the contorograph profile marked A in *Fig. 3* be reduced to a 1 : 1 scale, the actual rail profile is as shown in *Fig. 5*. It would be better if new rails could be rolled to the profile they ultimately achieve in service, but *Fig. 5* is asymmetric and therefore unsuitable. The symmetric profile shown in *Fig. 6* attempts to reproduce the normal worn profile on both sides of the apex, and is the one suggested for new rails. The essential difference from the existing British Railways flat-bottom profile (shown dotted in *Fig. 6*) is the provision of 2-inch intermediate radii between the top radius and the two corner radii.

Lillie [5] suggested that " the section . . . which will give the least wear is that section which will retain its shape during wear—i.e. the shape which the wear is tending to produce." He showed by geometric means that these

conditions were satisfied by a tractrix curve. This is the curve traced by a point D (*Fig. 7*) when taken in tow by an object F passing along the line AB, the constant length of the tangent DF being the length of the tow-line.

In *Fig. 7* the line AB is parallel to the axle. But rails are normally inclined at 1 in 20 to the axle, and the contorograph profiles are based on

Fig. 3

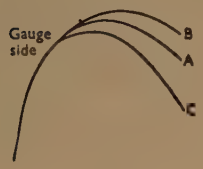


Fig. 4

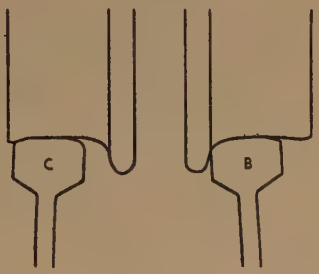


Fig. 5

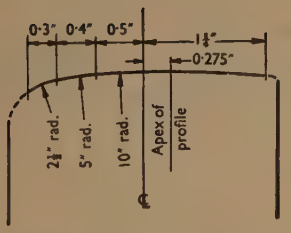


Fig. 6

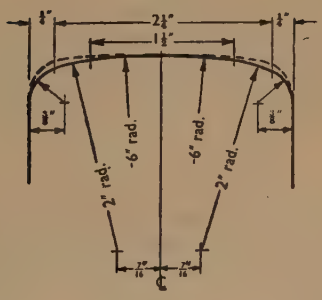
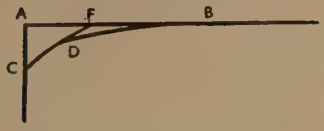


Fig. 7



the rails. Profiles of a number of worn rails have, therefore, been recorded with the contorograph tilted at an angle of 1 in 20 to the rail, so as to be based on a line parallel to the axle. These special profiles were then re-drawn with the horizontal as well as the vertical scale enlarged 10 times actual size, and corrected for the circular path of the contorograph pen. The curve so obtained was found to agree closely with a tractrix described

by a 5-inch tangent (0·5 inch actual rail scale) for a distance of $2\frac{1}{4}$ inches from the gauge side of the rail. This confirms Lillie's point and supports his suggestion that new rails should be given the normal worn profile.

(2) *Lateral Wear*

Although the contorograph gives no magnification of measurement in the lateral direction, it is possible to obtain a measure of the lateral wear from the lateral movement of the profile. This has been found to vary locally from nil to twice the vertical wear occurring during the first 9 years, even in straight track. This lateral movement of the profile is actually a widening of the gauge without movement of the rail, and without obvious side-cutting of the rail ; up to 0·35 inch has been recorded locally in straight track after the first 6 years.

It is not known how long this lateral movement continues, but observations on older rails which had been side-cut considerably showed that the lateral movement of the profile had been between 1 and $2\frac{1}{2}$ times the vertical wear. The profile of a high rail on a 17-chain curve traversed only by electric trains had a lateral movement of 1·8 to 2·4 times the vertical wear, and this was only slightly more severe than the ratio observed for straight track. However, rails on sharper curves, especially if subject to vehicles of long wheelbase, are known to suffer greater degrees of lateral wear than shown by these examples.

It is concluded that this lateral movement of the profile continues so long as tire flanges continue to press against the rail, and that it represents lateral wear of the rail and widening of the gauge, ranging locally from nil to 0·06 inch per year on straight track.

The Effect of Rail Composition and Heat Treatment

Rail-wear measurements are usually made to compare rails of different composition or heat treatment laid in the same track, and this type of investigation was pursued at several locations where such rails were known to exist or were specially laid. The composition was obtained from the cast analysis, or from a sample drilled from the rail web.

(1) *Basic-Open-Hearth and Acid-Bessemer Steel*

Rails rolled from casts of these steels having approximately the same composition (excepting nitrogen) were laid in the down line at Broadholme, between Derby and Ambergate, where they carried about ninety-two trains per day or about 1·7 million axles per year, of which about 30 per cent were passenger traffic. After $1\frac{1}{2}$ year had been allowed for running-in, comparative measurements were made at a total of twenty-two positions on four rails of each cast for a period of 5 years. The results of this experiment are summarized in Table 1.

Other rails already in existence in the same track at various distances

TABLE. 1.—COMPARISON OF BASIC-OPEN-HEARTH- AND ACID-BESSEMER-STEEL RAILS

Steel	Composition			Average annual loss in depth : inch	
	% C.	% Mn.	% P.	L.H. rail	R.H. rail
Basic open hearth . .	0·51	1·20	0·030	0·021	0·0235
Acid Bessemer . .	0·53	1·29	0·045	0·021	0·0235
	Nitrogen = 0·020%				

up to 1½ mile apart, and carrying the same traffic, were observed at the same time, and the results are summarized in Table 2.

TABLE 2.—COMPARISON OF BASIC-OPEN-HEARTH- AND ACID-BESSEMER-STEEL RAILS

Steel	Rail section	Composition			Average annual loss in depth : inch	
		% C.	% Mn.	% P.	L.H. rail	R.H. rail
Basic open hearth	95-lb. B.H.	0·51	1·05	0·037	0·022	
" "	" "	0·54	1·11	0·040		0·024
Acid Bessemer .	100-lb. B.H.	0·55	1·05	0·076	0·024	
" " .	" "	0·57	1·08	0·084		0·026
" " .	95-lb. B.H.	0·46	1·05		0·024	
" " .	" "	0·49	1·06			0·025
" " .	113-lb. F.B.	0·48	1·09	0·053	0·025	
" " .	" "	0·47	1·04	0·053		0·027

Tables 1 and 2 summarize the results of 5 years' measurements at twenty-four positions on basic-open-hearth and at twenty-eight positions on acid-Bessemer-steel rails of various compositions and sections, and all carrying the same traffic. When the rails were within a short distance of each other, as in Table 1, there was no difference in the rate of wear of these two kinds of steel. The small differences recorded in Table 2 are thought to be caused by variations in the local atmospheric conditions at positions as much as 1½ mile apart.

(2) *High-Carbon and Medium-Manganese Steel*

A comparison of four high-carbon bull-head rails of American origin with four British bull-head rails of British Standard medium-manganese composition was made in the down fast line about ¼ mile north of Watford Junction. The rails had been in the track 3·7 and 2·7 years respectively and their wear was measured at twenty positions for 3 years; during that period they carried about 85 trains per day, or 1·7 million axles per year,

of which about 90 per cent were express passenger traffic. Table 3 gives the results.

TABLE 3.—COMPARISON OF HIGH-CARBON- AND MEDIUM-MANGANESE-STEEL RAILS

Steel	Rail-web analysis		Average annual loss in depth : inch	
	% C.	% Mn.	L.H. rails	R.H. rails
High carbon	0·66	0·81	0·021	
	0·61	0·84		0·023
	0·60	0·75		0·023
	0·57	0·79	0·022	
Medium manganese	0·55	0·92	0·019	
	0·50	1·01		0·025
	0·49	1·04		0·023
	0·49	1·04	0·022	

There was virtually no difference between the wear of the two kinds of steel on the right-hand rail, whilst the slight difference on the left-hand rail was because of the greater wear of a " high "-carbon rail containing only 0·57 per cent carbon.

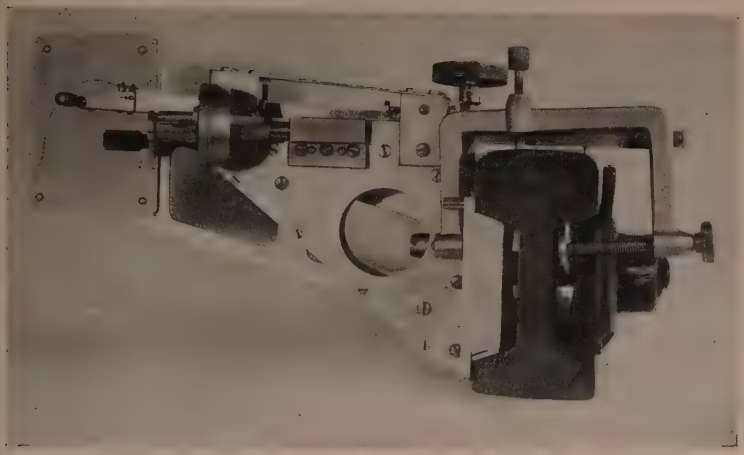
(3) *Sorbitic and Normal Steel Rails*

Sorbitic and untreated rails rolled from the same cast of basic-open-hearth steel (3542) were laid in the down main line near Darfield, between Sheffield and Leeds, which was then carrying about forty-eight trains per day, or 0·8 million axles per year. The composition and properties of these rails are shown in Table 4.

TABLE 4.—PROPERTIES OF SORBITIC AND NORMAL STEEL RAILS

Cast analysis	Acceptance tests	Untreated rail	Sorbitized rail	B.S. 9
%	*Falling-weight test :*			
C. 0·51	Deflexion after 7-ft drop	0·95 in.	0·80 in.	
Si. 0·14	„ „ 20 „ „	3·40 in.	3·15 in.	4·1 in. max.
Mn. 1·16				
S. 0·043	*Tensile test :*			
P. 0·053	Yield point, tons per sq. in.	26·4	47·2	
	Maximum stress „ „ „	54·0	64·8	46 min.
	Elongation on 2 inches	19%	18%	12/10 min.
	Reduction in area	36·4%	42%	
	Brinell hardness :			
	Average on cross-section	229		
	$\frac{1}{8}$ in. below running surface		321–335	

Fig. 1



THE SHAW RAIL CONTOROGRAPH

Fig. 10

After passage of:

52 axles

108 „

124 „

216 „

228 „

530 „

CONTACT BETWEEN WHEELS AND RAILS AS REVEALED BY WHITE PAINT

Sorbitic and untreated rails of acid-Bessemer steel were found to have been included in the same re-laying, so measurements were made on these also. After an 8 months' running-in period wear measurements were started and continued for 2·7 years.

TABLE 5.—COMPARISON OF SORBITIC AND NORMAL STEEL RAILS

Steel	Composition			Average annual loss in depth : inch	
	% C.	% Mn.	% P.	L.H. rail	R.H. rail
Basic open hearth cast 3542					
Untreated . . .	0·51	1·16	0·053	0·015	0·018
Sorbitized . . .	0·51	1·16	0·053	0·013	0·016
Acid Bessemer cast 333					
Untreated . . .	0·47	1·16	0·059		0·016
Sorbitized cast 142 .	0·48	1·00	0·065	0·015	

The sorbitized rails wore about 12 per cent less than the untreated rails of the same cast. Within 2 weeks of being laid there was a distinct difference in the appearance of the two kinds of rail, the sorbitized rails showing minute bright spots irregularly distributed over the running surface, whereas the untreated rails were normal. During the first 2 years the pattern on the sorbitized rails gradually changed to one consisting of two discontinuous bright martensitic streaks and nodules, each about $\frac{3}{4}$ inch from the rail centre-line.

(4) *Grain-Controlled Steel*

In spite of their very low Izod impact value of 2 to 4 foot-lb., steel rails are generally well able to survive the repeated impacts they receive at joints and crossings. This may be because rail steel is not brittle in the absence of a notch, since an unnotched Izod specimen will absorb the full 120 foot-lb. by bending without breaking. However, rail failures usually have their origin in some kind of crack or flaw which may be a manufacturing defect, a fatigue flaw, or a crack caused by the slipping or sliding of wheels. Any of these is equivalent to a notch and is likely to cause sudden fracture if present in a region of high tensile stress. It is, therefore, desirable to seek means of increasing the toughness of rail steel, but not at the expense of any other property.

O'Neill[8] described some attempts to achieve this by heat treatment, but lack of success led him to conclude that the low notch impact-resistance of ordinary rail-steel was caused by its inherent coarse-grain size in conjunction with its carbon content. He, therefore, asked for one ingot of a basic open-hearth cast (G.K. 284) to be treated with aluminium to promote

a fine inherent grain size for comparison with normal coarse-grain rails from the same cast.

Three rails of each kind were laid in the down main line near Darfield at the same time as, and adjacent to, the sorbitic rails mentioned in (3) above. The composition and properties of these rails are shown in Table 6.

TABLE 6.—PROPERTIES OF UNTREATED AND GRAIN-CONTROLLED STEEL RAILS

Cast analysis	Acceptance tests	Untreated rail	Grain-controlled rail	B.S. 9
%	*Falling-weight test :*			
C. 0·56	Deflexion after 7-ft drop	0·85 in.	0·80 in.	
Si. 0·14	„ „ 20 „ „	3·05 in.	2·95 in.	4·1 in. max.
Mn. 1·07	*Tensile test :*			
S. 0·032	Maximum stress, tons per sq.in.	56·8	57·6	46 min.
P. 0·023	Elongation on 2 inches	16·0%	12·5%	12/10 min.
Check analysis	**Laboratory tests**			
%				
C. 0·55	Izod impact test : ft lb.	4, 4, 4	7, 8, 9 8, 10, 10	
Si. 0·14	Diamond hardness : HD/100	220–254	233–254	
Mn. 1·08	Inherent grain size : A.S.T.M.	2	7	
S. 0·033	Actual grain size : A.S.T.M.	4–5	4–5	
P. 0·014				
Ni. 0·18				
Cr. 0·08				

The fracture of the tensile test piece was coarser in the grain-controlled than in the untreated rail. The actual grain size was the same, and the micro-structures were similar.

Wear measurements were started 8 months after laying and were continued for 2·7 years. Rails from other casts in the same piece of re-laying were included in the measurements for the sake of comparison.

There was practically no difference in the rate of wear of the grain-controlled and the untreated rails of cast G.K. 284, nor between the untreated rails of different analysis. (See Table 7.)

(5) *Special Rails at Chadwell Heath*

A large experiment in rail wear was initiated in 1943 by the former L.N.E.R. in the down line out of Liverpool Street, London, involving seven casts of acid Bessemer steel of normal and special compositions some being in the sorbitic condition. Wear measurements were not started until after the war, and traffic amounted to about ninety trains per day or 1·3 million axles per year, mostly steam-hauled express passenger trains

TABLE 7.—COMPARISON OF UNTREATED AND GRAIN-CONTROLLED STEEL RAILS

Steel	Composition			Average annual loss in depth : inch	
	% C.	% Mn.	% P.	L.H. rail	R.H. rail
Cast G.K. 284	0·56	1·07	0·023		
Untreated				0·012	0·013
Grain-controlled				0·013	0·014
Untreated rails :					
Cast G.M. 4433	0·58	1·00	0·065	0·015	
Cast G.P. 1677	0·51	1·02	0·025	0·014	
Cast 3542	0·51	1·16	0·053	0·015	0·018
Cast 333	0·47	1·16	0·059		0·016

ith a few freight and multiple-unit electric trains. A crossing within the est length introduced an additional fifteen trains per day and 0·26 million xles per year to some of the rails, but these made no significant difference o the rate of wear of rails of the same composition.

Measurements were continued for 2 years, and fourteen to twenty easuring positions were generally used for each type of rail steel. Eastern egion locomotives have no continuous blow-down and there was no ifference between the wear on the left-hand and right-hand rails.

All the sorbitic rails had developed surface irregularities and some were emoved before or during the measurement period for this and other easons. The results of this experiment are summarized in Table 8.

Sorbitizing has given most benefit to the cast of steel having the highest arbon content (cast 75, 0·48 per cent carbon) reducing the wear by 28 per ent. This is a much greater improvement than was measured at Darfield ith rails of even higher carbon content (cast 3542, 0·51 per cent carbon). the sorbitic rails be excluded because of their corrugation and shelling, ere was not a great difference in wear between the various compositions ied. Apart from cast 55 which had the highest tensile strength and ast wear of the as-rolled rails, there was no clear relation between tensile rength and wear. The addition of about 0·4 per cent manganese or romium or both, to the usual medium-manganese steel does, however, duce the wear slightly (5 to 15 per cent).

Special Rails at Kibworth

These 95-lb. bull-head rails were laid in 1932 on a 100-chain curve on e up line between Leicester and Market Harborough where the traffic ounts to about seventy trains per day, or 1·7 million axles per year, of ich about 30 per cent were passenger traffic. Contorograph measure-ents were made on the high rails only and the results are shown in Table 9.

TABLE 8.—SPECIAL RAILS AT CHADWELL HEATH ARRANGED IN ORDER OF RESISTANCE TO WEAR

Cast No.	Condition	Composition				Tensile test			Average annual loss of depth : inch
		C.	Si.	Mn.	Cr.	Max. stress tons/sq. in.	Elong. %	Redn in area : %	
75	Sorbitic	0·48		1·05		64·6	14·0	33·6	0·013
55	Normal	0·45		1·43	0·44	59·2	17·5	33·6	0·015
43	Sorbitic	0·44		1·48		65·2	12·0	33·6	0·016
53	Normal	0·40		1·02	0·42	49·6	22·0	46·0	0·017
105	Normal	0·51		1·53		58·0	18·0	32·0	0·017
106	Normal	0·49		1·52		57·6	17·0	27·6	0·017
43	Normal	0·44		1·48		53·2	20·5	39·2	0·018
53	Sorbitic	0·40		1·02	0·42	77·6	8·0	18·4	0·018
75	Normal	0·48		1·05		50·8	21·0	36·4	0·018
71	Normal	0·44	0·72	1·14		53·6	24·0	40·4	0·019
B.S. 9 acid Bessemer medium manganese		0·40/0·50	0·10/0·30	0·90/1·2		44 min. (over 50	12 min. 10 min.)		

Note : Except where indicated, the silicon content was 0·12% to 0·16% ; sulphur, 0·03% to 0·05% ; and phosphorus, 0·05% to 0·06%.

TABLE 9.—SPECIAL RAILS AT KIBWORTH

Description	Composition			Brinell hardness	Average annual loss in depth : inch
	% C.	% Mn.	% Cr.		
Low chromium	0·57	0·85	0·50	229	0·014
Higher manganese	0·62	0·98		247	0·015
Higher carbon	0·63	0·68		207	0·0165
Lower carbon	0·52	0·68		208	0·017
Low chromium low carbon normalized	0·44	0·80	0·44	234	0·020

Although situated on a 100-chain curve on which express passenger trains travel at a high speed, the lateral movement of the profile on the high rails averaged only 1·2 times the vertical wear, which value is often exceeded locally in straight track.

There is clear evidence here of the increased wear resistance provided by increasing carbon, manganese, and chromium. Raising the manganese from 0·68 per cent to 0·98 per cent has had a greater effect than raising the carbon from 0·52 per cent to 0·63 per cent. It is unfortunate that none of the compositions represented medium-manganese steel to B.S. 9, but this had not then been introduced. There is no direct relation between the Brinell hardness and the wear resistance of this group of rails.

(7) *High-Phosphorus-Steel Rails*

There have been many claims that phosphorus increases resistance to wear, and during 1936 a small cast of steel containing 0·2 per cent phosphorus was made to investigate this. To counteract the well-known embrittling effect of phosphorus the carbon was reduced to 0·26 per cent and additions were made of copper and manganese. The ingot was rolled into 95-lb. bull-head rails which had the properties shown in Table 10.

The two 60-foot rails were laid in the up fast Rugby–Euston line at about 160 yards from the exit of Watford Tunnel, and two pairs of normal medium-manganese rails were laid either side of them for comparison. The track is straight and in a deep cutting where the corrosive fumes, drawn out of the tunnel by the trains, persist. Wear measurements were made by the L.M.S. Civil Engineer's Department for 7·1 years, using a micrometer. Traffic averaged about eighty-five trains per day, or 1·7 million axles per year, of which about 90 per cent were express passenger traffic.

The high-phosphorus rails wore 6 per cent more on the average than the medium-manganese rails. (See Table 11.)

TABLE 10.—PROPERTIES OF HIGH-PHOSPHORUS-STEEL RAILS

Cast analysis	Mechanical tests	B.S. 9 medium-manganese basic-open-hearth steel
%	*Falling-weight test :* Deflexion after	
C. 0·26	7 ft. drop = 1·0 in. (1st blow)	
Si. 0·20	20 ft „ = 3·6 in. (2nd blow)	4·1 inch max.
Mn. 1·32	20 ft „ = 6·1 in. (3rd blow)	
S. 0·024	20 ft „ = 8·7 in. (4th blow)	
P. 0·20		
Cu. 0·62	*Tensile test :*	
	Maximum stress = 49·2 tons per sq. in.	46 min.
	Elongation on 2″ = 25%	12/10 min.
	Reduction in area = 48%	
	Izod impact test :	
	Head = 3, 2, 2 ft-lb.	
	Web = 2, 3, 4 „	
	Foot = 4, 7, 4 „	
	Unnotched = 117 „ at + 20° C.	
	„ = 66 „ at − 15° C.	
	Diamond hardness : 221	
	Brinell „ : 200	

TABLE 11.—COMPARISON OF HIGH-PHOSPHORUS AND MEDIUM-MANGANESE-STEEL RAILS

Steel	Average annual loss in depth : inch	
	L.H. rail	R.H. rail
High phosphorus . .	0·0350	0·0355
Medium manganese .	0·0315	0·0350

THE EFFECT OF RAIL DESIGN

(1) *Flat-Bottom* versus *Bull-Head Rails*

The increasing use of flat-bottom rails brought a request for a careful comparison of their rate of wear with that of bull-head rails in the same track. Suitable sites were found on the up and down fast lines at Cricklewood where the rails had already been in the track for 8 to 12 years.

On the up fast line traffic amounted to fifty-seven trains per day, or 0·8 million axles per year, mostly express passenger. Measurements were made for a period of 3 years at eight positions on each kind of rail, and the results are summarized in Table 12.

TABLE 12.—COMPARISON OF FLAT-BOTTOM AND BULL-HEAD RAILS (UP LINE)

Rail section	Width of head : inches	Composition		Average annual loss in depth : inch	
		% C.	% Mn.	L.H. rail	R.H. rail
131-lb. F.B.	3	0·56	1·07	0·019	0·020
110-lb. F.B.	$2\frac{7}{8}$	0·54	1·05	0·016	—
110-lb. F.B.	$2\frac{7}{8}$	0·51	1·05	—	0·016
95-lb. B.H.	$2\frac{3}{4}$	0·51	1·14	0·018	—
95-lb. B.H.	$2\frac{3}{4}$	0·48	1·11	—	0·018

In spite of their extra width of head, the 131-lb. flat-bottom rails have not worn more slowly but have actually lost about 20 per cent more in section than the narrower 110-lb. and 95-lb. rails. Distributing the wheel load over a head wider than the usual $2\frac{3}{4}$ inches is therefore not a means of reducing rail wear.

The corresponding site on the down fast line was situated between the platforms of Cricklewood station and on a slight curve of 130 chains radius. In spite of the moderate train speeds (about 40 miles per hour) and the slight curvature, there was considerable flange wear on the high (R.H.) rail which necessitated the turning of the high rails after wear measurements had been under way for slightly more than one year of the 3 years involved. Traffic on this line amounted to about eighty trains per day or ·06 million axles per year, of mixed traffic. The results of 3 years' measurements at four positions on each kind of rail are summarized in Table 13.

TABLE 13.—COMPARISON OF FLAT-BOTTOM AND BULL-HEAD RAILS (DOWN LINE)

Rail section	Composition		Average annual loss in depth : inch	
	% C.	% Mn.	Low L.H. rail	High R.H. rail
110-lb. F.B.	0·54	1·01	0·023	—
110-lb. F.B.	0·55	0·99	—	0·023
95-lb. B.H.	0·51	1·12	0·024	—
95-lb. B.H.	0·54	1·08	—	0·024

Here again, there is no significant difference in wear between flat-bottom and bull-head rails.

Comparisons between flat-bottom and bull-head rails are also given in Table 2. In this case the 113-lb. flat-bottom rails ($2\frac{3}{4}$-inch width of head)

wore slightly more than various bull-head rails within 1½ mile in the same track, but this is thought to have been caused by the humid atmosphere associated with the trees lining the flat-bottom track.

(2) *Rails with Welded Inserts*

The irregular formation and subsequent cracking and shelling of martensitic nodules from sorbitic rails led O'Neill[8] to experiment with metals other than rail steel applied to the running surface in the form of continuous strips or inserts. One rail received an insert of 60/40 brass applied as an oxy-acetylene welding deposit to a recess, 1½ inch wide by ⅛ inch deep, machined along the centre of the running surface ; and another rail was given a similar deposit in a recess, ¾ inch wide by ¼ inch deep, with the sides undercut to ⅞ inch wide at the bottom to give better adhesion of the brass to the rail. Austenitic stainless-steel weld metal was deposited by manual electric-arc welding on to a third rail having a 1¼-inch-wide-by-¼-inch-deep recess machined along the centre of the rail head, and pre-heated to about 350° C. before welding.

The weld deposits on all three rails were machined flush with the surrounding steel before being put in service on a 60-chain curve in the up slow line at Whitmore, between Crewe and Stafford, where the traffic amounted to about thirty-seven trains per day, or 1·2 million axles per year—mostly freight trains running at about 20 miles per hour.

After a running-in period of at least 6 months, wear measurements were started and continued for 2 years. A normal rail of the same cast, and other rails which had already been in the track 20 years, were measured for comparison.

The results are summarized in Table 14, in which the wear is expressed both as loss of section and as loss in depth because these two are not proportional to each other in all the rails. The effect of the curve and the difference between the left-hand and right-hand rails have proved to be negligible in this case.

TABLE 14.—WEAR OF RAILS WITH WELDED INSERTS

Type of insert	Average annual loss of section : sq. inch		Average annual loss of depth : inch	
	L.H. low rail	R.H. high rail	L.H. low rail	R.H. high rail
60/40, brass, 1½ in. wide		0·079		0·029
60/40 brass, ¾ in. wide		0·075		0·032
Stainless steel, 1¼ in. wide	0·017		0·004	
Normal rail of the same cast	0·044		0·019	
Normal rails, 20 years old	0·042	0·040	0·015	0·017

Laboratory wear tests[8] had previously shown 60/40 brass to be particularly wear-resistant (about 400 times more) compared with rail steel when rubbing against tire steel. Moreover, the work-hardening capacity as measured by the Meyer "*N*" index of 60/40 brass and of austenitic stainless steel was higher than for rail steel, and therefore suggested improved resistance to wear. Both materials also resist atmospheric corrosion better than rail steel.

The track tests have shown that the presence, in the centre of the rail, of a brass insert, even if only $\frac{3}{4}$ inch wide, will increase the rate of wear by 80 per cent. This is because the lower elastic modulus and yield point of brass causes the pressure of the wheels to be transferred to the margins of the running surface, where there is little or no lateral support to the steel and, therefore, more rapid wear.

The austenitic stainless steel, on the other hand, wore so little that it became proud of the surrounding margins of rail steel; its wear was only 40 per cent by section and 20 per cent by depth at the centre compared with that of a normal rail. The cost of applying stainless steel to a rail by this method makes it a very uneconomic proposition. An alternative would be to apply a weld deposit of stainless steel to a billet before rolling down to rail section. Steelmakers were consulted but they were very doubtful if this would produce a satisfactory rail.

The Effect of Corrosion on Rail Wear

The rapid wear of rails in tunnels traversed by steam locomotives is notorious, being accelerated by the corrosive action of the steam and sulphurous gases on the running surface. Steel is also liable to corrosion in the open air, although at a reduced rate compared with tunnel conditions. A compact and adherent layer of rust can retard considerably the initial corrosion rate of bare steel, and most rails which have been in the track in rural districts for up to 20 years are still true to their original width when measured by calipers after removing loose rust from their unworn surfaces. Frequent traffic prevents the accumulation of such a protective layer on the running surface, particularly along the centre, where a relatively bright strip usually develops in the course of time. Wear of the running surface thus exposes it to rapid corrosion and the question arises: if wear accelerates corrosion, does corrosion accelerate wear even in open-air conditions?

The absence of steam and sulphurous gases from the tunnels of the electrified Euston–Watford line provided the opportunity for comparing rail wear inside and outside these tunnels. The measurements were made near the exits of the tunnels (single-line bores) to ensure that the rails inside were never wet by moisture carried in by the wheels on wet days (this is confined to about 20 yards from the entrances). The measuring positions inside the tunnels were between 11 and 35 yards from the exit;

the rails at the exit were measured at positions inside and outside the tunnel, thus enabling comparisons to be made on the same rail. The rails immediately outside the tunnels were found to wear slightly less than those farther away; this was ascribed to the temporary drying of the former on damp or wet days by the dry wheels and dry air emerging from the tunnel with each train.

Measurements were continued for $1\frac{1}{4}$ to 4 years, and the results are summarized in Table 15.

TABLE 15.—WEAR OF RAILS INSIDE AND OUTSIDE TUNNELS ON THE ELECTRIFIED EUSTON–WATFORD LINE

Tunnel	Exit	Position	Traffic		Average annual loss in depth: inch	Reduction
			Trains per day	Axles per year		
Primrose Hill	Down	Outside	95	770,000	0·009	—
" "	"	Inside	"	"	0·005	45%
" "	Up Euston	Outside	60	480,000	0·016	—
" "	" "	Inside	"	"	0·009	44%
" "	Up Broad St.	Outside	35	290,000	0·013	—
" "	" "	Inside	"	"	0·010	23%
Kensal Green	Down	Outside	200	1,800,000	0·012	—
" "	"	Inside	200	"	0·006	50%

The up line divides on leaving Primrose Hill tunnel, and the Euston and Broad Street branches are curved at the exits. This feature accounts for their greater wear compared with the down-line exit which is on the straight and carries the combined traffic.

In all cases there is a marked reduction in wear inside the tunnels where the rails are not exposed to rain, although exposed to humidity, so that atmospheric corrosion is still possible. This was demonstrated by 4-inch square panels of mild steel which were exposed near the measuring positions for 430 days and suffered a loss of 10 to 11 grammes outside the tunnels, and 3·5 to 4·1 grammes inside.

The tube lines of the London Transport Executive were next considered as a suitable subject for the study of rail wear in the absence of corrosion, since the relative humidity and temperature in the tubes are about 50% and 75° F. during the summer, and 45% and 67° F. during winter. Corrosion of bright or rusty steel does not occur below 65 per cent relative humidity.[9] Facilities were kindly provided for caliper measurements of rails on straight track inside and outside tube tunnels, at positions well removed from exits or stations; and these measurements were supplemented by others provided by the Executive. The wear was based on the assumed depth of the rail when new, and the rails in question had been in the track for periods of 8 to 24 years. Because of the approximate method used, there were considerable variations in the measurements which could not

be related to any differences in traffic, which amounted to between 240 and 375 trains per day, or 2·3 to 3·6 million axles per year. The average annual loss in depth was 0·008 inch ± 30 per cent in the open air, and 0·003 inch ± 50 per cent in the tubes; that is, a reduction of about 60 per cent in the tubes.

The annual wear on the London Transport Executive's lines, even in the open air, was considerably less than on British Railways' lines carrying less traffic. This may be because the static tare load on the driving axles of motor bogies is only 10·8 tons in the tubes compared with 14 tons on the Euston–Watford line; but since the wheel diameters of tube motor-bogies are only 36 inches and 40 inches when new, compared with 43 inches on the Euston–Watford line, the Hertzian stresses[10] (that is, the maximum compressive stress at the centre of the area of contact between the wheel and a rail having a transverse radius of 9 inches on the running surface) are 67 to 68 tons per square inch in the former and 72 tons per square inch in the latter case. This small difference in contact stresses appears insufficient to account for the big difference in wear.

Nevertheless, the very low rate of wear inside the tubes is associated with dry conditions in which corrosion is absent, and the remarkable wear resistance exhibited by a rail having a welded insert of stainless steel is obviously related to its corrosion resistance.

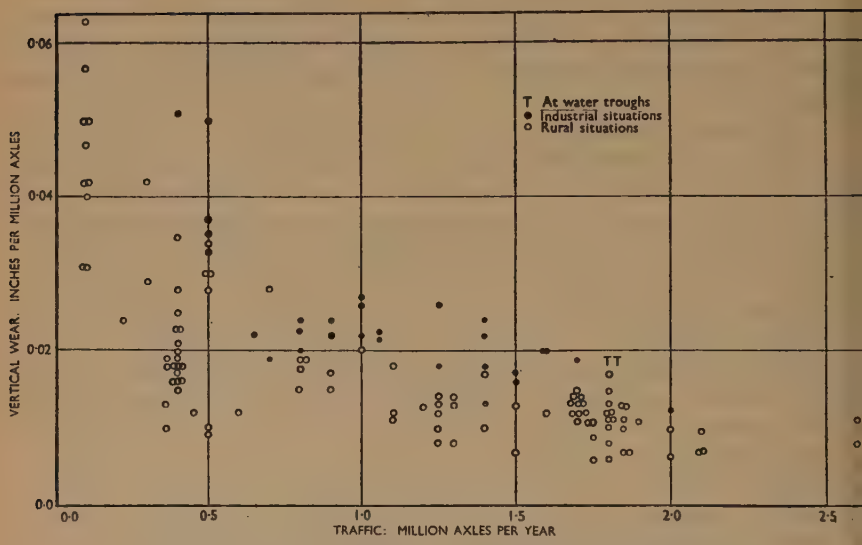
The Relative Effect of Traffic and Corrosion

Traffic has been assessed by the number of axles passing, and this was estimated from sample counts of trains per week and of axles in different types of train, and subsequently counted by an electro-magnetic axle counter developed by the Engineering Division of the British Railways Research Department. Since wide variations in wear not ascribed to composition were found on the same line or on other lines carrying similar traffic, it was decided to supplement the contorograph measurements with others made by calipers and covering a wider range of conditions. The combined results for steam-operated lines are plotted in *Fig. 8* which shows that rails in industrial situations and on water troughs wear considerably faster than in rural situations.

Hudson and Stanners[11] have shown that in Great Britain the sulphur pollution of the atmosphere is a major factor in the rate of corrosion of steel, so that corrosion may be up to 2½ times as rapid in an industrial area as in a rural situation. The Author[9] found that in a midland industrial atmosphere only 35 to 40 per cent of the total corrosion of steel could be ascribed to the direct effect of rainfall, that a further 35 to 40 per cent was caused by humidity on dry days, and the balance was ascribed to the effects of drizzle, fog, and dew, and to prolonged drying after rainfall had ceased. Rainfall is thus a relatively minor factor under these conditions.

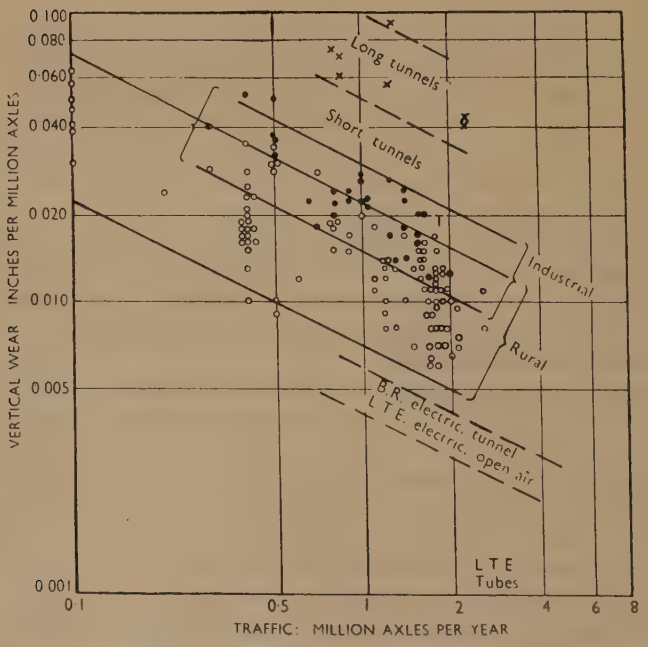
It is probable, therefore, that the variations in wear for the same annual

Fig. 8



WEAR OF RAILS PER MILLION AXLES—STEAM OPERATED LINES

Fig. 9



WEAR OF RAILS PER MILLION AXLES, VARIOUS CONDITIONS, STRAIGHT TRACK

traffic in rural or industrial surroundings is because of the local variations in atmospheric pollution and humidity now known as " micro-climates."

The influence of corrosion is also seen in the high wear per million axles at low traffic levels when the loss from corrosion is spread over very few axles, whereas at high traffic levels the opposite applies. If wear were directly proportional to axles the wear per million axles would be approximately the same at all traffic levels.

The relative effect of traffic and corrosion under most of the conditions found in Britain can be seen in *Fig. 9* which is *Fig. 8* plotted on logarithmic scales with the addition of values for tunnels on steam and electrically operated routes. In the absence of corrosion, or if corrosion had no effect on wear, the lines would be horizontal. If corrosion were entirely responsible, the lines would be inclined at 45 degrees to the horizontal. They have been drawn at 30 degrees and it is submitted that they agree reasonably with the upper and lower limits for steam traction in industrial and rural conditions, but there is insufficient evidence to show whether this relation also obtains in the more extreme conditions of tunnels and tubes. The 30-degree lines represent a relation of the kind :

$$\text{Wear per million axles} = \frac{K}{\sqrt{\text{millions of axles per year}}} \quad \text{inch}$$

$$\therefore \text{Wear per year} = K\sqrt{\text{millions of axles per year}} \quad \text{inch}$$

TABLE 16.—VALUES OF K FOR STRAIGHT TRACK

Location	Atmosphere	Traction	K
Tunnel	Steam and smoke	Steam	0·05–0·1
Open air	Industrial	Steam	0·015–0·03
Water troughs	Water splash	Steam	0·025 approx.
Open air	Rural	Steam	0·007–0·022
Open air	Suburban	Electric (B.R.)	0·010–0·018
Tunnel	No steam or smoke	Electric (B.R.)	0·004–0·006
Open air	Suburban	Electric (L.T.E.)	0·004–0·006
Tube tunnel	Dry and clean	Electric (L.T.E.)	0·0015–0·0025

Approximate ranges of the value of K for different conditions are given in Table 16. They represent the relative annual wear under these conditions for the same number of axles per year, and they extend for an extreme range of about 50 to 1. For British Railways' lines in the open air the range is about 4 to 1, equivalent to a sixteen-fold range in axles per year, so that 125,000 axles per year in the worst industrial conditions would cause the same annual wear (0·01 inch) as 2 million axles in the best rural conditions.

Examination of the Running Surface

Many main-line rails exhibit a relatively bright strip, ½ to 1½ inch wide, bordered by bluish-grey margins of crushed rust, which suggests that the majority of wheels make contact only along a narrow strip of the running surface. This was studied by applying thin coats of quick-drying white paint to the rail between trains, as illustrated in *Fig. 10* (facing p. 464). Observations at many places, and including rails not exhibiting a bright strip, have given the impression that in general at least 90 per cent of the wheels make contact within a strip about 1 inch wide, and that up to five hundred wheels may have to pass before all the running surface has been contacted.

In spite of this intensive use of the centre of the rail, it does not wear any faster than the lightly-used margins, since the profile normally remains unchanged. With equal wear per year, the wear per million axles at any spot across the width of the rail is inversely proportional to the axles per year contacting that spot; this would be represented on *Fig. 9* by a line inclined at 45 degrees, which suggests that corrosion is entirely responsible for the wear of the margins of the rail.

The rates of wear of the margins and the centre are probably inter-related; if one wears a little faster it concentrates more axles on to the other until a balance is achieved. Then the wear of both is governed by that of the heavily used centre strip. To what extent is that accelerated by corrosion?

The condition of the running surface was studied with the aid of a bench microscope fitted with a dismountable camera. *Figs 11* to *14* were recorded between trains on a busy line carrying about ninety trains per day and towards the end of a prolonged spell of fine weather during August and September 1947. After 36 days without rain, these frequently used rails were still dull grey in appearance, showing that corrosion by humidity had continued in the absence of rain and had prevented the traffic wearing the rails completely bright and clean.

The observations were made at or near the centre of the rail, and all that could be seen at first was a mass of crushed rust. After rubbing lightly with cotton wool moistened with alcohol, small areas of bare steel became visible (*Fig. 11*). Light rubbing with 00 emery paper revealed more steel, but still extensively riddled with corrosion pits (*Fig. 12*). More vigorous cleaning still left many pits in the steel (*Fig. 13*), even in a bright streak of martensite (*Fig. 14*).

The insulating effect of this layer of crushed rust on track-circuited rails is well known to signal engineers.[12] The conversion of rust (Fe_2O_3) by pressure into black magnetic oxide of iron (Fe_3O_4) was recognized by Spring[13] in 1888.

This pitted condition of the running surface of rails in the open air cannot be regarded as ideal for resisting wear when compared with rails

in protected situations such as the tube tunnels, where they are able to develop a solid work-hardened wear-resisting surface over their full width.

Conclusion

The information presented suggests that the wear of rails on the frequently used open-air lines of B.R. is greatly dominated by atmospheric corrosion of the running surface. Since wear and corrosion mutually accelerate each other, it is difficult to assess their respective influences accurately, but an inspection of *Fig. 9* suggests that in the case of a busy line carrying 2 million axles per year the wear of the rails is increased by a factor of 3 in the best rural conditions and by a factor of 12 in the worst industrial conditions, taking the London tubes as a basis.

Even more speculation is needed at the lower end of the traffic scale where 100,000 axles per year represents an infrequently used branch line and the factors are of the order of 12 and 50 respectively.

Since the measurements supporting this conclusion were made on straight or slightly curved track, it may be thought that they are not applicable to sharper curves on which there is considerable lateral wear. But the side of the rail is just as exposed to corrosion as the top, and it has already been shown (" Some Features of Rail Wear," section (2)) that lateral wear up to twice the vertical wear (in inches, not square inches) is quite common locally in straight track, and such positions have been included in the measurements. Sharp curves are inevitably short in length and therefore only a very small proportion of the total track mileage. A detailed survey of 70 miles of main line in the Midlands showed that 84·5 per cent was straight or curved with a radius greater than 80 chains. A rough survey of over 1,000 miles of main and secondary lines throughout Britain, using 1-inch ordnance survey maps, showed that more than 80 per cent was straight or curved with a radius greater than 100 chains. The Continental railways, with the exception of Germany, Switzerland, Austria and Norway, generally have less than 10 per cent of their track curved with a radius less than 25 chains.[14] Many foreign railways are of the opinion that lateral wear of the outer rail only becomes particularly noticeable on curves of less than 25 chains radius on standard gauge.[15] Lateral wear on curves is already being reduced by lubrication.

With these facts in mind, it is considered that the wear measurements and conclusions expressed in this Paper are typical of more than 90 per cent of B.R. track.

The dominant effect of corrosion explains the difficulties experienced in distinguishing between the wear of rails of different compositions, although this might be easier on sharp curves. What is needed for most of the track is a corrosion-resisting and wear-resisting rail which does not depend on a compact and adherent layer of rust for its corrosion resistance. Additions of copper and phosphorus are often made to increase corrosion resistance,

but they have produced no beneficial effect in a rail (see "The Effect of Rail Composition and Heat Treatment," section (7)).

Metal-sprayed coatings of various thicknesses of zinc and aluminium have been applied to the running surfaces of rails after thorough cleaning by shot blasting under ideal conditions in a workshop. It was hoped that the passing wheels would pressure-weld the sprayed metal to the rail, but the resulting adhesion was insufficient for railway conditions, and the protective coatings peeled off during the passage of the first few trains.

A welded insert of stainless steel has shown remarkable wear and corrosion resistance, but it cannot be applied economically.

Until a suitable corrosion-resisting rail steel is forthcoming, increased rail life may be obtained because of the greater strength of flat-bottom compared with bull-head sections, and by pursuing standards of design, manufacture, and maintenance which will reduce the liability to failure and so permit rails to remain in service for longer periods than at present.

Acknowledgements

The Author wishes to thank Mr T. M. Herbert, M.A., Director of Research, British Railways, and Mr C. E. Dunton, M.A., M.I.C.E., Chief Civil Engineer, London Transport Executive, for permission to publish the information contained in the Paper; and Mr J. D. Swindale, B.Sc., and other members of the Metallurgy Division of the British Railways Research Department for assistance in the work.

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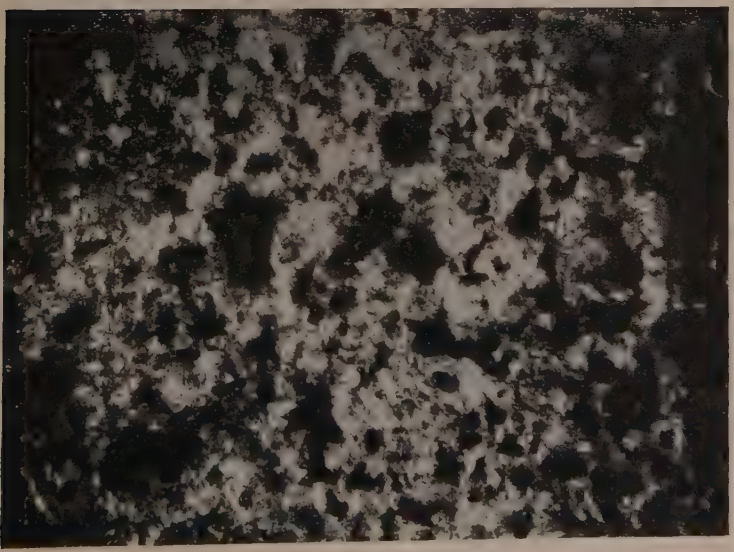
Fig. 11

UNNING SURFACE OF RAIL AFTER CLEANING WITH COTTON WOOL AND ALCOHOL
(Magnified one hundred times)

Fig. 12

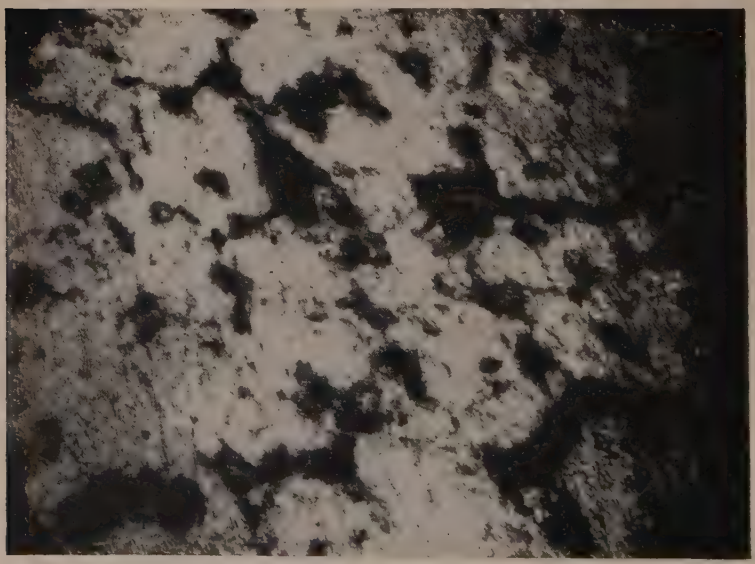


RUNNING SURFACE OF RAIL AFTER LIGHT CLEANING WITH OO EMERY PAPER
(Magnified one hundred times)

Fig. 13

Running Surface of Rail After Vigorous Cleaning with OO Emery Paper
(Magnified one hundred times)

Fig. 14



Running Surface of Rail, Showing Bright Martensitic Streak, After Vigorous Cleaning with OO Emery Paper
(Magnified one hundred times)

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The Paper is accompanied by six photographs and five sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

Mr A. K. Terris observed that, as the Author had indicated, there were many Papers on wear of steel rails and suggested that Mr Dearden's Paper should be read with those as a background. Previous Papers had indicated clearly the desire of civil engineers for a tougher rail, commensurate with cost, and their willingness to try anything that the metallurgist or steel expert was prepared to suggest. Where they failed, he thought, was that, despite much work and testing, insufficient data had been obtained by the measurements they could make upon which to form clear judgement. It was only the modern technique and instrumentation of the research engineer that had made it possible for the present Paper to be written.

Whilst it had been realized that the chemical composition was one of many factors governing rail life, there had been a lack of precise data on which to base any comparisons. The Paper made a very practical contribution to data on measurements and showed, in fairly certain terms, the relative effect of traffic and corrosion on the head wear of rails ; it was also important in the way it brought out the great effect of corrosion on the loss of weight of a rail.

The most corrosive conditions on railway track occurred in tunnels ; it was there that loss of weight took place most quickly. It was known that, by corrosion alone, rails lost weight in the web and in the foot where there was no mechanical wear. It was, however, in the head of the rail that the loss occurred most quickly, and that was the portion of the rail affected by the passage of traffic. Having regard to that well known fact, it was strange, therefore, that engineers had not apparently given greater cognizance to the importance of corrosion in rails in various atmospheric conditions in the open.

On p. 478 the Author referred to the intensive use of the centre of the rail by the wheels and postulated that, since a rail maintained its profile, it seemed that wear on the margins arose entirely from corrosion. Mr Terris wished to ask him whether it would be too much to attribute such

wear to what was now called "fretting corrosion." Mr Terris made no claim to be an expert on the subject, but, as a practical engineer, he had been attracted by what he had read recently in regard to the subject, and he thought that might have some application to the circumstances arising out of the interaction of wheel and rail. That seemed to mean that the worn off virgin metal became oxidized by friction and formed hard abrasive particles and chemical compounds ; it appeared possible for those particles to be displaced under the action of the wheel from the centre towards the edge of the rail, which might account for the profile maintaining its shape despite the greatest pressure being in the centre of the rail.

That factor could, no doubt, be related in some measure to the Author's suggestion that the radius of the top table of the rail should be 6 inches. The Author's proposal, however, seemed exceptional having regard to modern trends in the development of the rail section. The radius of the crown of the 95-lb. bull-head rail was 12 inches which, in the 109-lb. flat-bottom section rail, had been reduced to 9 inches, and at present there was a suggestion to maintain the flatter radius of about 12 inches at the centre and introduce a sharper radius of 3–3⅝ inches towards the edges. The Author, however, suggested going down to 6 inches but Mr Terris did not know what effect a 6-inch radius at the centre of the head would have on the stress elements in the rail itself.

Unlike previous Papers and despite the Author's very wide survey of the problem and comparison of many different kinds of rail steel, the Paper did not suggest any change in the composition given in the present B.S. specification. Could it be assumed that railway civil engineers were complacent about the present chemical composition of steel used and there were no improvements possible in that respect ? There appeared, however, to be indications in the Paper of possibilities along certain lines ; it would be interesting to hear more about them and particularly if the narrow limits of chemical composition were justified.

In dealing with the test rails at Chadwell Heath, it was stated that there was no clear relation between the tensile strength and the wear of rail, and it was further stated that the wear of the rails at Kibworth was not directly proportional to the Brinell hardness. From those statements did the Author conclude that the tensile and Brinell tests gave no measure of the wear resistance of rails in the track, and, if so, would he recommend some other form of mechanical test, such as a laboratory wear test, for that purpose ? Finally, had the Author any suggestions to make about how research could be directed towards overcoming the defects of what had been at one time the favoured sorbitic rail ?

Mr T. M. Herbert paid tribute to Mr W. K. Wallace, the Chairman of the Division, for being the inspirer and originator of much of the work described in the Paper when it had first been started on the former London, Midland, and Scottish Railway.

Mr Herbert said there was one point which he wished to make ; it con-

cerned the question of relating wear to traffic and the criteria which should be adopted in assessing traffic. The Author took as his measure the number of axles passing over the rail, which seemed to imply the assumption that every axle did an equal amount of damage. Mr Herbert was not at all sure that that was true. Particularly when dealing with what was agreed to be a combination of wear and corrosion, it might very well be that perhaps the engine and the first few vehicles—that was, the first few axles—did all the damage.

Whether it was the number of axles or trains that conditioned the wear, there was also speed, which had not been mentioned, and axle-load, which might have some significance. Those were additional factors which the Author might well examine and on which research might be conducted. Speed and axle-load might be very important, in view of possible electrification and the fitting of freight trains with continuous brakes, to permit much faster running. It was important to know the possible effect of those developments on rail wear.

Dr F. T. Barwell said that in approaching the question of whether wear accelerated corrosion or corrosion accelerated wear, it was desirable to be a little more precise in the definitions used. If he was right in thinking that the Author's definition of wear was the total loss in depth of rail which he measured, and his definition of corrosion was the existence of a chemical reaction, it was easy to conceive a situation in which a rail surface corroded and became oxidized, the passage of a wheel removed that oxide, more oxide was then formed, and the sequence continued; if the conditions promoted rapid oxidation, there would be a definite relation such as the Author instanced between wear and corrosion. If, however, wear was simply defined as the amount of material removed from the surface by some mechanical action, and corrosion as the amount removed by chemical action, the question took a rather different form, and it might, indeed, be possible to put forward the hypothesis that corrosion might be, in some circumstances, a wear-preventing process.

From the Paper the answer might be given that in tunnels on London Transport railways, where there was no obvious corrosion, there was very little wear, but it seemed necessary to look more carefully into the nature of the oxide which formed. It was well known, for instance, that stainless-steel inserts were corroding in the sense that they formed an oxide film, but it was a very durable film. The need for a little more care in characterizing the nature of the surface product seemed to be emphasized by the Author's reference on p. 478 to the work of Spring in 1888. Dr Barwell had not had an opportunity to check the original reference, but he felt that the statement made in the Paper was rather improbable; he did not think that in 1888 sufficient had been known of the different oxides and hydroxides which could be formed—there were at least six—whilst the techniques for their identification by X-ray, electron diffraction, and so on did not then exist.

That led him to feel that his earlier illustration of rusting occurring between trains, and the trains clearing the rust away, might be far from the truth. A possible hypothesis might be that the pitting shown in a slide by the Author had been formed by a micro-cell corrosive action ; but, as Mr Terris had said, the work on fretting corrosion had indicated that there were possibly much more subtle relations between wear and corrosion, and that, indeed, the debris might, to put it crudely, act as an abrasive which, in combination with the oxidized surfaces and bare metal, might also set up electrolytic cells giving rise to that pitting type of corrosion. It seemed, therefore, that a very much more detailed study was needed of the mechanism of oxide formation and metal removal from the surface of the rail.

From consideration of the Paper, it might be imagined that pure metal-to-metal contact and removal of material did not occur. It was well known, however, that one of the most widely accepted theories of friction sought to explain it, and did explain it in a great many cases, by the fact that the two surfaces became welded together and then pulled away. The Paper seemed to indicate that that mechanism was not working to a great extent in rail traction, so that some other mechanism must be sought to explain what went on at the contact between wheel and rail ; Dr Barwell thought that that would be just as revealing in explaining variations in coefficients of adhesion as he hoped that it would be in explaining variations in wear.

In conclusion, he wished to express the hope that a very much more detailed study would be made of the actual mode of the removal of metal. He thought that, as Mr Terris had indicated, a very interesting correlation would then be found with the experience of those who were working on such phenomena as fretting corrosion.

Mr C. J. Allen observed that some surprising statements were made in the Paper, beginning with the statement in the Synopsis that " No difference could be detected between acid-Bessemer and basic-open-hearth-steel rails of the same nominal composition, nor between high-carbon- and medium-manganese-steel rails." The second of those statements was doubtless based on Table 3, but it was very difficult to regard the compositions given there as fully representative of the difference between medium-manganese and high-carbon steel. The first of the medium-manganese rails mentioned might actually have been in either category, with no more than 0·92 per cent manganese and with 0·55 per cent carbon, whilst among the high-carbon rails there was one with no more than 0·57 per cent carbon. If a true comparison were desired between the characteristics of rails of high-carbon and medium-manganese composition, rails should have been selected which were more towards the upper limits in each case, so that there was a distinct difference between the two qualities. He wondered, therefore, whether Table 3 could really be regarded as a fully satisfactory basis of comparison.

On p. 469 the Author remarked that "Raising the manganese from 0·68 per cent to 0·98 per cent has had a greater effect than raising the carbon from 0·52 per cent to 0·63 per cent." Mr Allen thought that those two increases would be regarded in the steel world as roughly proportionate, and there it was definitely stated, notwithstanding the sentence in the Synopsis which he had read, that the manganese had had a markedly better effect so far as wear was concerned. If the sentence quoted from the Synopsis gave a correct impression it might lead to doubt whether the entire trend of the present day from the high-carbon rail towards the medium-manganese rail was based on a complete misconception and that would be very hard to believe. With manganese there was more freedom from rolling defects and possibly less risk of transverse fissures than with high-carbon rail, but beyond that there must surely have been some justification for the change from high-carbon to medium-manganese rails which had taken place in the United Kingdom and other countries.

No reference at all had been made in the Paper to the wear of manganese-alloy rails containing 12 to 14 per cent of manganese, notwithstanding the increasing use of manganese alloy in locomotive work, for instance in liners for axleboxes. Perhaps the Author could give an indication of why—possibly it was on economic grounds—manganese alloy had apparently gone out of use for rails.

Another aspect of the Paper which seemed to call for question was the very short duration of the periods of rail-wear measurement, in some cases no more than 2 years. Was it to be assumed that those measurements could be regarded as representative of the wear during the full life of the rail? What about the possible effects of work-hardening as a gradual process in increasing wear resistance? Mr Allen wondered how much reliance could be placed on deductions drawn from tests of such limited duration as those described.

Some extremely interesting experiments with high-phosphorus-steel rails were described on pp. 469–470. Although the Author had obviously confined himself to the wear of contemporary rails, Mr Allen recalled the rails that had been rolled in the earliest decades of steel manufacture and the phenomenal life which some of them had shown in the track, in some cases 40 or even 50 years, right into the 1920's. The Author had laid great emphasis on the dominating influence of corrosion in wear. If that were so, how was it that that dominating influence had not been seen in loss of weight by the rails in question, during so extremely lengthy a period?

He wondered, in relation to the experiment with high-phosphorus-steel rails which had been described, whether the high percentages of phosphorus, sulphur, and manganese in those early rails, which would have secured their instant rejection on present-day specifications, did actually have some effect not only in increasing resistance to wear but also resistance to corrosion. So far as concerned the experiment with high-phosphorus steel rails which the Author described, it seemed a pity that it had been so timid,

and that the carbon content of the small cast which had been made was no more than 0·26 per cent. The situation might have been saved to some extent by the 1·32 per cent of manganese, but, even so, could very much increase in wear resistance be expected with the carbon at that level ? Mr Allen felt that that interesting experiment might well be continued, and that, with phosphorus between 0·1 and 0·2 per cent, it might be possible to risk, say, 0·40 per cent carbon at least. Another possible line of experiment, in view of the invariable combination of high sulphur and high manganese in the early years to which he had referred, might be in the direction of percentages of sulphur and manganese above the limits regarded as normal at the present time. He did not share the opinion that the possibilities of variation in the composition of steel used for rails, with a view to securing better wear, were now at an end.

It was evident from the Paper that, from the point of view of wear, the sorbitic rail came out well. In Table 5 there was 12 per cent less wear with the sorbitic than with the untreated rails of the same cast, notwithstanding the fact that the sorbitized acid-Bessemer cast contained only 1 per cent of manganese, whereas the untreated cast with which it was compared contained 1·16 per cent of manganese. In Table 8—in which he was particularly interested, because he had been the representative of the London and North Eastern Railway at the works at which those rails were made—there had been the even more substantial advantage of a 28 per cent reduction in wear with one of the sorbitic casts. An even better figure might have been recorded but for the unfortuate fact that the wear tests had not been begun until between 3 and 4 years after the rails had been laid in the track.

With regard to the irregular surface hardening of sorbitic rails, to which reference was made, did the Author consider that that was definitely the result of the sorbitic structure, and, if so, how would he explain the development of the same defect with non-heat-treated 1-per-cent-chromium rails, which had been observed in at least one test on the former London and North Eastern Railway ? Had the Author any information whether similar trouble was being experienced in the United States with the heat-treated rails which were currently being laid in large numbers ?

In travelling through Germany Mr Allen had noted " roaring " rails extending for considerable mileages, for example, on the the main line from Cologne to Basle, which he thought might be martensitic rails from the Bavarian works at which that quenching process was applied. The Germans were introducing mobile plants for grinding out those corrugations in situ.

Did the Author consider that a reversion, in Britain, to heat treatment of rails by the sorbitic or any other process, even though it led to some irregular surface hardening, would be justified if some economical method of rail-grinding could be devised ?

In his conclusion the Author had no specific remedy to offer to improve

wear on rails except the substitution of flat-bottom for bull-head rails, and the production of rails with less liability to failure than those in current production. Mr Allen suggested that the possibilities of improved composition, and especially of heat treatment were not yet exploited to the fullest advantage.

Mr D. H. Coombs referred to the correlation between the Hertzian stresses and rail wear mentioned on p. 475 and said that the type of wear which the Author was there considering was plain head wear, but the types of wear with which the excessive Hertzian stresses were most commonly associated were one form or another of surface defect. Mr Coombs did not feel that it was reasonable to suggest that there was much correlation between head wear and Hertzian stresses, because with high Hertzian stresses there was work-hardening of the surface, and with a work-hardened surface the head wear was usually reduced, though the incidence of other surface defects in the rail was usually enhanced.

Excessive Hertzian stresses were generally associated with those defects which the Americans comprehended in the term " shelly " rail, and that was the aspect of big loads on small wheels which at least for London Transport had been mostly in focus. Although it was true to say that recent American work on the photoelastic analysis of rail stresses had suggested that the Hertzian analysis, as developed in 1930 by Thomas and Hoersch on the assumption of continuing elasticity of the rail steel, was of doubtful validity, to the extent that the relations deduced by them were quantitatively in need of adjustment, it appeared that qualitatively their conclusions still stood, and if the conclusions of Thomas and Hoersch on Hertzian rail stresses and surface deformation did hold, then Mr Coombs thought it was logical that so far as rail surface defects were concerned, there was or there should be a correlation between the tensile strength of the rail and its useful life. It might well be that that did not hold for plain head wear, but it seemed from American opinion that it might well hold for other defects.

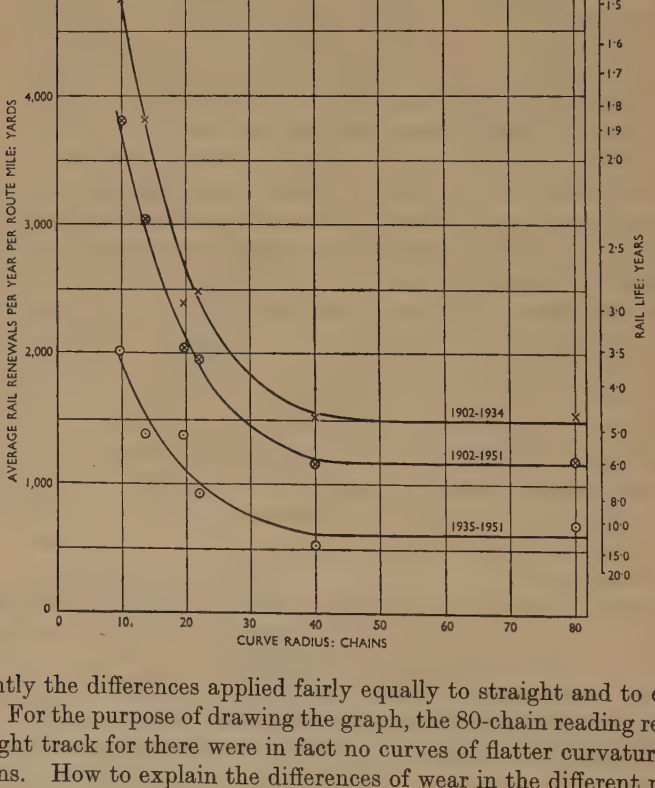
However, if the problem of " shelly " rails was considered, whether from the point of view of elastic work done or of plastic work done, there was an equal chance of coming to the view that a static-load (tons)/wheel-diameter ratio in excess of about 0·2 was not desirable for too many of the wheels passing over any particular piece of track.

Other points which seemed to indicate the need for caution in using the Hertzian approach of Thomas and Hoersch, and subsequent similar work, including recent photoelastic studies, were that none of them took into account bending stresses, braking stresses, traction stresses, or other dynamic increments.

If, however, rail wear was considered from the economic point of view some sort of historical approach should be adopted, because head wear was probably for London Transport only a minor cause of rail renewal. There were other wear factors, such as sidecutting, surface defects, chair gall, and

so on ; in integrating all factors the study of past records was useful and, though it had considerable limitations, it indicated some of the difficulties in construing past experience of rail renewals in relation to the wearing properties of rails. *Fig. 15* showed rates of rail renewal on a section of the District Line (of London Transport System) for the three periods 1902–34, 1902–51, and 1935–51. It would be seen that, for a given length of track there were three different rates of wear for three different periods and that

Fig. 15



apparently the differences applied fairly equally to straight and to curved track. For the purpose of drawing the graph, the 80-chain reading referred to straight track for there were in fact no curves of flatter curvature than 40 chains. How to explain the differences of wear in the different periods was a problem with no obvious answer.

Mr L. N. Flatt said that the Author, having touched briefly on the question of cost, went on to emphasize the dominant effect of corrosion ; possibly he would be prepared to go a stage further. Table 14 and the related data were most instructive. Did the Author feel that he had sufficient information available, or, if not, did he think that it would be possible to obtain it, to justify the preparation of a curve which might be

regarded as reasonably reliable and which would show the relationship between an increase in the resistance of rails to corrosion and the increase in price which it would be economical for users of rails to pay for that increased resistance ? That was a very important economic factor, and Mr Flatt felt that if the Author could give at least some information on that point it would be valuable because if steelmakers had such data, it would guide them on the extent to which they could go in experimenting to obtain a rail to give improved service by withstanding corrosion for a longer period.

Professor Hugh O'Neill observed that Mr Dearden had justified his main conclusion that the loss in depth of rails of standard compositions was dominated by atmospheric corrosion, and thanks were due to him for condensing his multitudinous results into the expression on p. 477, and for giving values of the " loss coefficient," *K*.

Regarding the influence of different compositions, it should be remembered that only a relatively narrow range had been examined. It was well known that a steel rail with a pearlitic microstructure resisted wear much better than one of wrought iron ; therefore, there must be an optimum amount of carbide constituent and structure to give good wear. That generalization held good, since there were no important differences in the atmospheric corrosion of steels containing different amounts of pearlite—though sorbite probably rusted faster than pearlite. He was glad, therefore, to read on p. 469 that " there is clear evidence of the increased wear resistance provided by increasing carbon, manganese, and chromium." Manganese and chromium both went to form carbide, and could be given coefficients of equivalence in terms of carbon as pearlite-producing elements. In that respect manganese might be divided by some figure such as three to give the equivalent carbon effect. Results for a range of composition had been extracted from the Paper and were listed in Table 17.

TABLE 17

Table	Location	Composition			Loss : inch
		per cent C.	per cent Mn.	$C + \frac{Mn}{3}$	
10 (p. 470) .	Watford Tunnel exit	0·26	1·32	0·70	0·035
2 (p. 463) .	Belper	0·48	1·09	0·84	0·025
3 (p. 464) .	Watford Junction	0·66	0·81	0·93	0·021
1 (p. 463) .	Broadholme	0·53	1·29	0·96	0·021
8 (p. 468) .	Chadwell Heath	0·51	1·53	1·02	0·017

Improved performance appeared to correspond with increased carbide content.

Professor O'Neill had always doubted whether the various current

standard rail compositions gave appreciably different rates of wear ; that was one reason why the subject was studied afresh. Mr J. Shaw was approached to learn whether any of his standard and sensitive wear instruments could be applied to rail measurements. Instead, he produced the ingenious rail contorograph which had performed satisfactorily.

In the 7 years which had elapsed since his association with the Author, Professor O'Neill wished to ask whether the finest possible reading from the contorograph records was the 0·0005 inch given in Table 11. In Table 5 the sorbitized rails were said to be 12 per cent better than the untreated rails because of a loss difference of 0·002 inch. Equal discrepancies in Table 2 were dismissed as " small differences " and Table 7 was in the same position. Would the Author care to comment on that matter ?

Incidentally, the experiment with which Professor O'Neill had been associated regarding inherent grain-size steels was chiefly intended to study cracking resistance. It had hardly been expected that there would be any significant difference in wear. Similarly the austenitic rail inserts (Table 14) were designed to deal with a surface exfoliation which was believed to develop from surface martensite produced by wheel-slide near a signal. In fact they had confirmed that material with high work-hardening capacity and high corrosion resistance gave good service life. Perhaps the Author could state the composition of the weld metal or of the austenitic electrodes used.

If rails with corrosion-resistant top surfaces could not be made economically from " pluramelt " ingots (as Professor O'Neill had previously suggested in Reference 8) or by " chromizing," then the question arose whether a corrosion inhibitor might be applied from the guard's van of each train.

Mr E. C. Cookson, expressing great interest in Mr Allen's remarks on the subject of progressive work-hardening under traffic, said that in 1900 the former Great Western Railway introduced a section of rail known as the OO, 97½ lb. in weight ; even in 1954 there remained some of those rails, put in up to 1905, in track with reasonable heavy traffic, on which wear was almost negligible. The Western Region still used 97½-lb.-rail in tunnels, but in the opinion of inspectors the present day OO was nothing like the old. He was tempted to ask whether the Author had investigated another aspect of the life of rails, namely, the effect of the conditions under which the rails were rolled in the mills. It might be suspected that formerly the rolling mills took more time and did not use quite such high temperatures, so that there might have been more work-hardening in the process of rolling.

The Paper was of great interest to those associated with the former Great Western Railway in one respect in particular ; that concerned the rail section recommended on p. 461. Following research in 1932 the Great Western Railway had come to the conclusion that that was the shape of rail which they ought to have, with the head radius, the intermediate radius,

and the corner radius exactly as recommended by the Author. Unfortunately, the permanent way engineer could not act alone in those matters and had to consult and to please the mechanical engineers, who would not allow the use of the 6-inch head radius, so that it had been necessary to go to 9 inches. They had actually rolled that 95¼-lb. rail in 1934. In 1938 they used the same head section and added $\frac{3}{16}$ inch to the head and produced 100-lb. rail, but unfortunately it disappeared when war broke out. He wondered why the result of experience of that kind was not reflected in some modern profiles.

There was very little reference in the Paper to the relation between tire wear and rail wear, and, since the wear on the rails had been taken at selected points during a period of years, it was the result of the accumulation of a number of different tire profiles passing over the one section of rail. Could additional research provide information on whether, if the mechanical engineers would keep their tires within the specified limits, it would be possible to get a better rail profile, and by such joint efforts stop hunting ?

*** **Mr D. R. Wattleworth** observed that the data presented would appear to indicate that increased hardness did not in itself result in better resistance to wear. If that was so it was curious that the conclusion had not previously been drawn unless it was that the climate of the United Kingdom had provided favourable conditions for the observance of the effect of corrosion.

In other countries a great deal of attention had been directed towards the production of harder rails and in the United States much work was now being done with a view to improving rail wear. The indications were that whether by alloying or heat treatment the objective was greater hardness. With the varieties of climate and atmosphere encountered in that country it was curious that the influence of corrosion did not appear to have been observed.

The data on which the Paper appeared to have been based might seem to be rather scanty, but no doubt it would be extended in time. It would probably be of value to introduce some special compositions of rail steel into future trials with a view to finding something with better corrosion resistance. It would also probably be of interest to make a comparative test on a manganese-steel rail, in view of such steel having a high resistance to wear but a low resistance to corrosion.

Mr T. H. Turner observed that the measurements recorded by the Author appeared to show that rail wear was caused by the wheels rubbing off, and by vibration shaking off, the rust produced by more or less rapid corrosion in the open air.

For any given traffic intensity during the same time-interval rails wore least in a dry atmosphere, such as in the London underground tube railways.

*** This and the following contribution were submitted in writing upon the closure of the oral discussion.—Sec. I.C.E.

More wear occurred where rain or water troughs wet the rails. The worst rail wear was found in the sulphurous atmospheres of steam-engine tunnels.

The Author's figures agreed with what Mr Turner had written [16] in 1939.

> " The life of a rail in service depends on the nature and number of vehicles passing over it, the nature of the strata on which it is laid, and above all, on the nature of the atmosphere. Corrosion is one of the most important features in limiting the life of rails.
>
> The length of service life may be as little as two or three years in a tunnel, a dozen years or so in a stretch of line by water troughs, and as long as a man's lifetime in the clean air of rural conditions."

The new data, however, was quantitative whereas Mr Turner's earlier statements had been qualitative. That linking of rail wear and corrosion was what would be expected by corrosionists, who now knew that dry steel did not rust, and that wet steel rusted increasingly in proportion to the sulphur-dioxide content of the atmosphere.

The metallurgical research facilities available to British Railways were very limited and in directing them to rail problems, Mr Turner had to consider two major fields, in both of which the Author was now making very valuable contributions :—

(1) What action could be taken to prevent cracks and fractures which caused discontinuity of the track and might become catastrophic failures ? The British Railways Rail Failure Handbook, produced on the basis of the first draft which Mr Turner had prepared in 1947, had resulted in standardized rail-failure reports becoming available from all over the country for Hollerith card analysis and other such detailed examination.

(2) What could be done to minimize the gradual, but fairly uniform, wearing away of the rails in the track. That surface demetallization was caused by rusting and the rubbing away of rust, which was not the same phenomenon as occurred in abrasion tests in the dry air of a laboratory. The Author's contorograph and micrometer measurements were, therefore, of great value and should be continued.

Civil engineers had become more and more conscious of the losses caused by corrosion since the application of mathematics had reduced their metal thicknesses and factors of safety against corrosion.

The pioneering work of the Corrosion Committee of the Institution of Civil Engineers was followed by that of the Corrosion Committees of the Institute of Metals and of the Iron and Steel Institute.

The latter committee now functioned under B.I.S.R.A. and as the

[16] References **16** *et seq.*, are given on p. 505.

Chairman of its Atmospheric Corrosion Sub-Committee for the last decade, Mr Turner could state that no rail-steel compositions were known which gave markedly superior resistance to corrosion.

In the discussion on Mr Saniter's Paper[17] on railway steels, Mr Turner had said that the miles of rusting track were a challenge to metallurgists and he had questioned whether some sort of bronze-coated rail could not be developed.

Six years later, Mr Turner had become responsible for examining rails himself, as Chief Chemist and Metallurgist of the L.N.E.R., and Mr R. J. M. Inglis, M.I.C.E., who was then Assistant Engineer of the Southern area, had sent to him many old rails for examination in the laboratories. Those old rails had had excellent service records for 40 or 50 years in the tracks of secondary main lines carrying fairly heavy traffic, especially during the peak periods of summer holidays. They had not been taken from main line or suburban services, but had experienced service conditions comparable with that of the rails now in the tracks of the Southern Irish Railways, where, with light traffic and almost complete freedom from sulphur dioxide in the atmosphere, rails would serve from 40 to 60 years, despite heavy rainfalls.

In his early days on the Iron and Steel Institute's Corrosion Committee, Mr Turner had received reports from Mr Raymond Carpmael, M.I.C.E., Chief Engineer of the Great Western Railway, on comparative service tests on ordinary steel and on copper-bearing steel rails, which he had studied under the corrosive conditions existing in some of the G.W.R. tunnels. The copper-bearing steel rails had been definitely superior in the open air, but had not excelled in tunnels. Carpmael's tests were mentioned in the 4th Report of the Corrosion Committee,[18] but so briefly that the report gave the impression that a little extra life might be gained for rails by a small copper addition, although the difference would not be spectacular.

If all rails corroded about the same in the open air, and if Sir Hugh Beaver's Committee on Air Pollution resulted in no effective reduction in the sulphur-dioxide content of the atmosphere in industrial areas, what should railway engineers do as a result of the information provided by the Author ?

Could the rail makers produce aluminium-coated rails by metal-spraying the red-hot rail during its final pass through the mill ? If that were to prove possible, the fact that the aluminium was worn off the head of the rail would not spoil its assistance in protecting the foot and the web.

Even if there was little that could be done to retard the need for the removal of rails for rail-head wear, the need for the removal of rails for rail-end failures could be retarded by introducing long welded lengths. It might then be proved that long welded lengths, having less vibration, would wear better because the corrosion product was not so rapidly vibrated away.

In conclusion could the Author state whether anything was known

about the effect on tramrail-head wear of introducing vibration- and noise-reducing rubber-sprung wheel-centres. The problem was to devise a means of making the least-rubbed parts of the rail head corrode less ; the most-rubbed parts of the rail head corroded much less, because they were kept warmer, cleaner, and drier by the abrasion.

The Author, in reply, said he thought that Mr Terris was correct in suggesting that the wear of rails involved some degree of fretting corrosion. There was always a film of fine brown dust on the brightest part of the rail, and the abrasive nature of that dust might well accelerate wear. He could not say whether the thicker deposits on the less frequently used margins of the rail were an accumulation of wear products or of corrosion products ; in either case their abrasive properties might explain the rapid wear caused by the few wheels running on the margins.

There was a slight difference in electrical polarity between rusty steel and bright steel which would cause the former to corrode more rapidly than the latter when forming the components of a electrolytic cell. When the rails were wet, that effect probably intensified the corrosion of the margins, but not that of the relatively bright strip. The accumulation of rust on the margins would also harbour moisture which would promote slow but continuous corrosion in the absence of rain. All those features would promote rapid wear of the margins by the few axles which contacted them.

The proposed 6-inch radius over $1\frac{1}{2}$ inch of the crown (*Fig. 6*) would cause an initially high contact pressure between tire and rail, probably high enough to ensure quick relief by plastic deformation of the crown, which would thus be more severely work-hardened and better able to resist wear.

Professor O'Neill had shown that there appeared to be some connexion between wear resistance and composition. By inference, tensile strength and hardness should therefore be related to wear resistance, although other influences had appeared to obscure that in the narrow range covered by the Author's tests. He was not in favour of replacing tensile and hardness tests by laboratory wear tests, since they were no nearer to track conditions and were more difficult to control and reproduce.

Until some different composition had been tried out on a large scale, he would not suggest any change in the current B.S.9 and 11. Such trials might well include acid Bessemer rails in the same range of carbon content (0·50–0·60 per cent) as basic-open-hearth rails, whilst even higher carbon and/or manganese might be tried for the latter. The severity of the falling-weight test at present imposed by British Standards limited the development of harder rails, and therefore he suggested that a large quantity of such rails, conforming to a less severe standard in that empirical test, should be tested over a long period of years in the track.

There was a great need for a sorbitic rail which would wear smoothly, but he had no suggestion to make on that point.

Mr Herbert had discussed the criteria which might be used in relating traffic to wear. Since all tires suffered wear, the Author had assumed that all tires caused some rail wear, but he did not wish to imply that their effects might be equal. He had, therefore, preferred to relate rail wear to axles rather than to trains, since the average number of axles per train varied with the route. It might be that locomotives, especially those whose coupled wheels had worn to slightly different diameters, were responsible for much of the wear, but the greater the number of axles in the train the greater the chance of every portion of the rail running-surface being rolled over and not merely the central strip. American and Continental railways usually recorded traffic in terms of gross tons, and that was virtually equivalent to " number of axles " in the long run.

He had not yet been able to distinguish any effect of speed or axle load at the many locations tested. Owing to the very mixed nature of railway traffic it was difficult to find locations where either of them were substantially different and other factors the same. Low speeds were usually associated with braking, accelerating, or gradient climbing, and those factors were liable to increase the wear. There was no abnormal wear at locations where most of the traffic passed at 60 miles per hour or faster.

The Author's idea of the mechanism of removal of material from rails approximated to the first of Dr Barwell's definitions, but it should be remembered that the corrosion products were never completely removed, not even after ninety trains per day for 36 days without rain (*Figs 11–14*). The film of debris remaining on the rail might cause wear by abrasion and promote corrosion by adsorbed moisture; by preventing perfect metal-to-metal contact it prevented transient welding of tire to rail which might otherwise be a source of greater adhesion, but also of rapid mutual wear.

Spring's observation that intense pressure converted red rust into black magnetic oxide had been confirmed by Roberts-Austen,[19] and the Author had found the black scale on the rail running surface to be magnetic. However, he agreed that a closer definition of those oxides and hydroxides was necessary when using modern methods of identification.

In reply to Mr Allen, the Author thought that the compositions of the high-carbon rails in Tables 3 and 9 were representative, since B.S.9 specified 0·55–0·65 per cent carbon and 0·9 per cent maximum manganese for high-carbon rails of basic open-hearth manufacture. Whilst three of the four medium manganese rails in Table 3 were at the bottom of the range for basic-open-hearth steel of that grade (0·50–0·60 per cent carbon, 0·9–1·2 per cent manganese), the one in Table 9 contained 0·62 per cent carbon which put it above the medium manganese range and it was not surprising that it performed well. Although carbon was free and about three times as potent as manganese, the latter imparted valuable toughness to the steel. Steel containing 12–14 per cent manganese was still used

for switches and crossings where conditions were severe, but he had not made any measurements on them.

Although 2 years was the minimum duration of some of the tests, others had been continued much longer, but without getting any different result. In most cases the rails had been in service for several years before measurements commenced. On routes carrying 50–100 trains per day he would expect the rails to reach a maximum degree of work-hardening in less than a year, and he had found no increase in wear resistance after about the first 15,000 trains. He was of the opinion that corrosion-pitting of the running surface of rails in the open severely limited the degree of wear resistance which could be obtained from work-hardening in the absence of corrosion.

Regarding the long life of old steel rails, he had not heard of any which were not associated with an infrequent service of short trains in rural areas of low rainfall where corrosion was at the minimum. Under such conditions twenty trains of forty-five axles each per day would cause only 0·25 inch top wear in 50 years in straight track in the absence of wheel slip or slide, and it would be very difficult to distinguish between different rail steels in those conditions.

Whilst high phosphorus might increase resistance to corrosion, he would expect high sulphur to have the reverse effect, and both were undesirable from a mechanical point of view. The carbon content of the high-phosphorus rail (Tables 10 and 11) had been limited to 0·26 per cent to avoid undue brittleness in association with 0·2 per cent phosphorus ; but the tensile strength was still within B.S. limits for rail steel. The steel having 0·4 per cent carbon and 0·1–0·2 per cent phosphorus as suggested by Mr Allen would certainly be very brittle.

The tests summarized in Table 8 were commenced 8 years after the rails were laid only because their existence was unknown to the Author until 1951. By then, 0·10–0·14 inch would have been worn off the sorbitized layer and their condition would be nearer to the average than if they had been tested as laid with the original quench-hardened layer intact.

He was aware that non-heat-treated rails of various compositions were sometimes liable to develop surface roughness because of irregular hardening in service, particularly in Germany where about 46 per cent of the main lines were afflicted by it in 1951, but he did not know to what extent it occurred in the United States. He understood that martensitic rails were free from that trouble, being at the maximum hardness when laid and therefore unable to harden more, but they were not responsible for the widespread trouble in Germany. Although he had no data on the frequency and cost of rail grinding, he doubted whether the increased wear resistance of sorbitic rails would justify the cost of removing the hard nodules which might be partly responsible for that increased wear resistance.

In reply to Mr Coombs, the Author said that the Hertzian stresses had been quoted only as a basis of comparison of rail-wheel contact stresses on the electrified lines of British Railways and London Transport. The slight difference in contact stress did not explain the big difference in rail wear, but that did not imply that there was any correlation between them. He considered shelly rail to be caused by horizontal fatigue cracks which started just below the running surface at the plane of maximum shear stress due to wheel-rail contact. The large proportion of wheels imposing high contact-stresses made the corrosion-free and slow-wearing rails on London Transport lines more liable to that defect than the rails on British Railways lines where corrosion so accelerated the wear that the metal was removed before it reached the limit of that particular form of fatigue. Since head wear was only a minor cause of rail renewal on the London Transport system, the decrease in annual renewals of rails shown in *Fig. 15* was clearly attributable to a reduction in side-cutting or surface defects, and that suggested the beneficial effect of rail lubricators installed in the later part of the periods quoted.

Mr Flatt had asked the Author to enlarge on the economic aspect of rails. That was complicated by not knowing to what extent the cost of new sleepers and fastenings would be reduced by the use of more corrosion- (and, therefore, wear-) resistant rails, and whether the more expensive rails would yield a corresponding higher price when scrapped; the latter point was important because about 80 per cent of the original rail was involved. The Author felt, therefore, that he could give no reliable answer to Mr Flatt's question.

Professor O'Neill had listed in Table 17 the wear of rails at a few selected locations carrying roughly the same traffic, in an attempt to show a relation between wear and "equivalent carbon" $\left(C\% + \frac{Mn\%}{3}\right)$. The Author considered the difference in wear at some of those locations could be caused by differences in atmospheric corrosion, particularly the first, which was near a tunnel exit. He preferred to restrict such comparisons to individual locations such as Chadwell Heath and Kibworth, for which he had plotted annual wear against $C\% + \frac{Mn\%}{3} + \frac{Cr\%}{3}$ for the non-heat-treated rails. In spite of a considerable scatter of points there did appear to be a reduction in wear of approximately 10 per cent for an increase of 0·1 per cent in equivalent carbon within the range of 0·8–1·1 per cent on nearly straight track in rural situations, and it was probable that the reduction in wear would be more pronounced on sharp curves. By way of contrast, a change from average rural to average industrial conditions could double the annual wear. Other things being equal, a 0·1-per-cent increase in equivalent carbon should raise the tensile strength by 4 tons per square inch. On the other hand, recent measurements on

rails not mentioned in the Paper had shown no correlation between composition and wear, thereby reviving the Author's doubts on that matter.

The accuracy of the contorograph was about 0·001 inch, and the averages of a large number of measurements were reported to the nearest 0·0005 inch. The small difference of 0·002 inch was equivalent to 12 per cent of 0·016 inch in Table 5, and to only 8 per cent in Table 2 ; the latter case was not regarded as important because some of the rails were $1\frac{1}{2}$ mile apart and their wear might have been affected by local variations in atmospheric pollution and humidity, those showing the most wear being in sylvan (and therefore humid) surroundings. Nevertheless, the 95-lb. B.H. acid Bessemer rails in Table 2 were within 100–200 yards of the rails in Table 1 and situated under similar conditions, so that a valid comparison was possible. That showed that 95-lb. B.H. acid Bessemer rails (Table 2) of B.S. composition (0·40–0·50 per cent carbon) wore 10 per cent more on the average (taking left-hand and right-hand rails together) than basic-open-hearth rails (Table 1) of B.S. composition (0·50–0·60 per cent carbon). The acid Bessemer rails in Table 1 had been made to a higher carbon content than was allowed by B.S.9 so as to be comparable (in all but nitrogen) with the basic-open-hearth rails.

Although the welded inserts of 60/40 brass and austenitic stainless steel both possessed higher work-hardening capacity and corrosion resistance, it was only the latter insert which gave increased wear resistance. It had been suggested by Mr C. C. Inglis, of the British Transport Commission, that galvanic corrosion of the steel margin by the brass might insert have accelerated the wear.

The Author was interested to learn for the first time from Mr Cookson that the former Great Western Railway had recommended, though never adopted, the profile shown in *Fig. 6*. The alternative proposed by Mr Loach differed in detail but was the same in principle, that was to say, the introduction of an intermediate radius between the corner and the crown. The Author would agree to a 12-inch radius provided it was confined to the central $\frac{3}{4}$ inch of the crown. None of those dimensions were very critical since the profile of the new rail would soon wear in conformity with the tires, but it was desirable to have that condition with new rails to ensure smooth running and normal wear from the beginning. Because of the continual lateral movement of the tire on the rail, he thought there was little chance of the proposed 6-inch head radius wearing the tires correspondingly hollow, as was expected by mechanical engineers. Moreover, the high contact-pressures caused by a 6-inch head radius would soon be relieved by plastic deformation as mentioned in the reply to Mr Terris.

He had estimated that two to three times as much metal was removed from rails than from tires, including that removed from tires by braking and machining. If the latter were excluded, the ratio would be even greater. That was because of the greater total length of the rails and

he impairment of their wear resistance by corrosion owing to less frequent se than tires. There was a big field for joint research into tire and rail rofiles to promote smoother riding and reduced mutual wear.

The Author wished to assure Mr Wattleworth that the conclusions of he Paper were based on data which were not scanty. Each value of wear n Tables 1–15 was the mean of at least 2 years' measurements at eight r more positions on the rails concerned, and more than four hundred rofiles has been recorded during the Chadwell Heath experiment sum-narized so briefly in Table 8, whilst the data given in *Figs 8* and *9* were he reverse of scanty. He would agree that the Paper did not cover a very wide range of compositions, but that was limited by service require-nents; softer rails might have suffered from end batter, whilst harder ails might have been too brittle. As a result, the relation between wear esistance and hardness was often masked by the more marked influences f corrosion and traffic, but Professor O'Neill had pointed out that there vas some connexion between composition and wear.

The few rail-wear values the Author had obtained from the United States had come from high-speed straight-track locations in Illinois and Indiana,[20] and they were about one-third to one-half the values experienced n rural Britain for the same number of axles per year. In the United States the average axle load was about 2 tons heavier, the locomotive coupled axles about 50 per cent heavier, the carbon content of the rails bout 0·2 per cent higher; at the locations concerned, the annual rainfall vas about 30 per cent greater and the annual duration of high humidity over 80% R.H.) about half that in central England. Of those differences, he higher carbon content and the lower humidity were factors in favour f low wear. American freight cars were carried on 4-wheel bogies which night not be so liable to lateral hunting movements as British 4-wheel reight stock. American freight cars were fitted with cast-iron wheels aving chilled treads which were much harder than the tire steel used in Britain, and that feature should reduce rail wear according to the results f tests in Japan.[21] It was, therefore, not surprising that there were ractically no references to rail wear in American railway literature, lthough there was much concern over transverse fissures and shelly spots.

The Author agreed with Mr Turner that there was no immediate rospect of an economic rail steel which would resist corrosion on the elatively rust-free running surface. A steel containing 0·6 per cent opper had been tried in a moderately corrosive situation (Tables 10 and 11) ut with no obvious improvement. However, to obtain a true comparison was necessary to test rails rolled from the same cast of steel, part of hich had received an addition of 0·2–0·3 per cent copper.

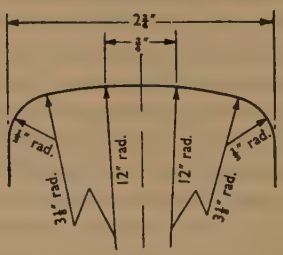
He had no knowledge of the effect of rubber-sprung wheel centres on he wear of tram rails, but he thought it might reduce the battering of he rail ends.

Correspondence

Mr J. C. Loach expressed interest in the profile of a rail head when new and suggested a correlation between the greater tendency on British Railways of carriage bogies with tires in an intermediate state of wear to hunt on newly laid track than on other track and the more rapid wear mentioned by the Author as occurring on the corners of the rail head than in the centre of new rails. The nominal 12-inch radius of the crown had been replaced by a nominal 9-inch radius in recent rail designs but although the increased tendency of bogies to hunt on the newly laid rails had been reduced, it had not been entirely eliminated and it was interesting to note that the Author had stated that with the smaller (9-inch) radius of the crown the rapid wear at the outer corner in the early stages of service had also been much reduced, though evidently not eliminated.

Mr Loach, therefore, agreed with the principle of the introduction of a smaller radius near to the sides of the running surface as the Author had suggested in *Fig. 6*, but he could not accept anything like so small a radius

Fig. 16



as 6 inches for the central part because of very high wheel-rail contact-stresses. Indeed he was proposing a reversion to the 12-inch radius for the central $\frac{3}{4}$ inch of the crown and then a $3\frac{1}{8}$ inch radius each side of it running into the $\frac{1}{2}$-inch radii at the corners as shown in *Fig. 16*.

A recent discussion with the Author had enabled Mr Loach to confirm that such a profile was remarkably similar to that which had produced the contorograph profile A in *Fig. 3* of the Paper.

The 12-inch radius of the central portion would be advantageous in reducing the wheel-rail contact-stresses again to what they were formerly and the restriction of that radius to the central $\frac{3}{4}$ inch of the crown would reduce the eccentricity of many loads borne by the rail and so reduce stresses in the webs and fillets during the early life of the rails.

Mr D. Herlihy, of Dublin, observed that rail wear under conditions of light traffic in rural Ireland was consistent with the values plotted for rural situations in the Paper. The wear per million axles as determined

from unmagnified profiles of worn rails was about 0·026 inch for traffic of approximately 100,000 axles per annum. Where traffic was 500,000 axles per annum the wear per million axles was about 0·011 inch. Those figures corresponded fairly well with those given in the Paper.

Those rates of wear gave values of 0·0082 and 0·0078 for K. The values were reasonably close and demonstrated that the Author's rule of the square root of the number of axles passing per annum held good for light traffic.

Dr Ing. Fritz Birmann, of Minden, Germany, observed that the measurements with the contorograph described in the Paper were clearly calculated to give a very accurate picture of the course of wear on the crown. It was a pity that lateral wear could not be equally accurately portrayed by the same means. For the latter purpose there was in use on the German Federal Railways ("DB") a feeler-type wear-measuring device, having six styles three of which bore against the sides of the rail head at any one time, giving readings which were accurate to within 0·1 mm. On the DB network lateral wear was a very important factor, because a large number of lines had a high proportion of curved track, and so the measurement of lateral wear on the rail head was carried out with particularly great care, and it had been found that the above-mentioned degree of accuracy was fully sufficient for practical purposes. The more pronounced wear in the first few months of use referred to in the Paper tallied with the observations made in the course of Dr Birmann's experience on the German railways, but the observation in regard to the more pronounced vertical wear indicated by the contorograph curve C in *Fig. 3* (opposite rail) was surprising. In his experience, in the case of track where the wheels ran with their flanges pressing hard against the rail (profile B in *Fig. 3*), more marked lateral wear began with 70 degrees tilt, as a result of sinusoidal travel of the vehicles, but in that case rail B was at the same time also subject to somewhat more pronounced vertical wear.

He thought that the ideal rail-head profile (*Fig. 6*) proposed in the Paper was applicable only to the conditions obtaining in Britain. Extensive measurements carried out in the past and in more recent years of the profiles of worn rails on German railways had shown that the radius of curvature was generally of the order of 350–370 mm. (13·75–14·5 inches). Since 1935 German rails had been rolled with a radius of curvature of 400 mm. (15·75 inches) and it had been found that after being in service for some time the rails acquired a mean curvature corresponding to a radius of approximately 360 mm. (14 inches).

Referring to Tables 1 and 2, p. 463, the fact that only negligible differences in wear had been observed between rails made from basic-open-hearth steel and rails made from acid Bessemer steel presumably arose from the small difference in C and Mn content between the contrasted sorts of steel. The same applied to the steels referred to in Table 3, p. 464. Furthermore, the differences in vertical wear were almost within

the limits of accuracy for track measurements. Greater differences would have been observed if the rails under comparison had been laid one behind the other in a curve on a stretch with high train-frequency and thus observed while subjected to identical conditions of service of an exacting nature.

In Germany comparative rail and rail material tests had been carried out for the past 25 years by laying the rails in selected test curves and keeping them under careful observation. Once a year, and if necessary at shorter intervals, those rails were measured at certain points in their length, namely in the middle and 1 metre from each end. The procedure had been described in detail in Dr Birmann's review of German rail problems.[22]

Referring to Table 5, p. 465, the corrugation of sorbitized rails observed in England indicated that elongation of the steel at the surface was altered by the sorbitic hardening and that the resulting unfavourable modification of the inherent properties of the material promoted corrugation which was invariably a matter of the ductility of the steel used, it being a recognized fact, for instance, that rails made from Thomas steel were more liable to become corrugated than rails made from open-hearth steel.

The sorbitized rails supplied to the German railways during the war by the Hayingen and Belval works had frequently proved prone to corrugation but had shown no sign of the shelling observed in the British sorbitized rails.

Martensitically hardened rails supplied by the Maxhütte works at Sülzbach-Rosenberg had proved very satisfactory, from the point of view of wear resistance, both on the German Federal Railways and in Switzerland, but at the same time, unfortunately, had shown, in part, some tendency to pitting. Martensitic rails were no longer being produced for the German railways. Tests were at present being carried out with induction hardening but the results of those tests were not yet available.

Turning to the special rails at Kibworth, mentioned in Table 9 on p. 469, Dr Birmann remarked that in Germany too it had been observed that higher manganese content increased wear resistance. The wear-resistant rails now generally used in Germany on main-line curves having a radius of less than 500 metres had a manganese content of 1·4–1·6 per cent. Heavy rails of cross-section S64 for laying in tunnels were also made from the same steel.

It was not surprising that the Author had found no relation between Brinell hardness and wear resistance, and that wear-resistance testing machines gave no clear indication as to the suitability of steels as rail material. Reliance could be placed only on tests carried out under actual working conditions, that was to say, on the behaviour of the rails traversed by wheels under load. The wear resistance of welded joints could likewise be tested only in working conditions.

A particularly noteworthy feature of the Paper was the section on tests

with inserts welded into sorbitic rails. There could be no doubt that that expedient would affect corrugation and shelling, but it was presumably too costly to be considered in practice.

The section in the Paper headed "The Effect of Corrosion on Rail Wear" (p. 473) contained interesting statistics on electrically operated traffic through tunnels in England (Table 15), but it would have been still more interesting to have had comparable figures for steam-operated tunnelled sections of line. On the German Federal railways, rail wear in long tunnels with steam-operated traffic was three to four times as great as in the open. In the long Schlüchten tunnel, for instance, which had a length of 3·6 kilometres, the wear amounted to 2–2·5 mm. (0·08–0·1 inch) compared with 0·4 mm. (0·013 inch) per year on the same line in the open.[23]

It had been found that the smoke issuing from tunnels had a powerfully corroding effect on the rails for a distance of 50 metres from the tunnel mouth.

Rails laid in tunnels with steam-operated traffic on the German Federal Railways were coated with anti-rust media in accordance with normal practice since 1949, for which purpose it had been found that two priming coats of thinned red lead followed by a top coat of acid-proof bituminous protecting paint gave satisfactory results. Other iron parts of the permanent way in tunnels were not galvanized, since that procedure had proved its value in the past.

Dr Birmann believed that the extremely low annual wear of 0·008 inch on open underground railway tracks was attributable to the relatively low axle loads and travelling speeds of the trains in question.

It was equally true of conditions on the GFR that rails were subject to higher corrosion in industrial areas, for example, in the Ruhr district, than in rural areas, although the question of the inter-relationship between corrosion and rail wear had not yet been systematically investigated in Germany.

The observation had also been made in Germany that rails and steel sleepers had an extremely long life in country districts. Thus, for instance, Bessemer-steel rails from the Maxhütte works, supplied in the year 1871, were still in use on a branch line near Bayreuth and were in relatively good condition.

The Author had reported that on British Railways annual rail wear increased with the square root of the number of axles and did not therefore increase linearly in proportion to the increase of traffic. GFR experience, on the other hand, was that the dependence of wear on traffic density was almost linear. There might be several reasons for that; for example, the generally higher axle loads on all goods trains in Germany, which caused metal particles to become removed at a uniform rate on the wearing surface of the rails. It would appear that with less dense traffic and longer intervals between successive trains, the steel molecules were able to "recuperate"

(in accordance with more recent theories of wear put forward by Professor Fink and others). In conditions of lower traffic frequency combined with lower axle loads, such as on branch lines, experience had shown that the rails had time to undergo a process of cold hardening at the surface, whilst, in German experience, rails carrying dense and heavy traffic showed a practically linear wear curve. It would be interesting to know what experience had been gained on those matters in Switzerland and Austria on mountain railways.

The Author thanked Dr Birmann for his valuable communication. The longer radius of curvature found on the heads of worn German rails suggested that the average tire profile and the lateral motion of the wheels might be different on the German from those on British Railways. The compositions of the rails in Table 1, p. 463, were made intentionally similar in order that a true comparison between steel-making processes might be obtained.

The Author appreciated that greater differences in wear would have been observed if the rails had been tested on sharp curves, but he had preferred straight or slightly curved track as being more typical of British Railways as a whole. There was also difficulty in finding a curve sufficiently long and of constant cant and radius so that all the test rails were exposed to the same severity of wear.

Although sorbitized rails in Britain nearly always became corrugated in service, only a minority of them developed shelling.

Values of rail wear in steam-operated tunnels were given in *Fig. 9*, and Dr Birmann's figures for annual wear inside and outside such tunnels were within the range of values indicated in *Fig. 9*, assuming 1 million axles per year. The paint system recommended by Dr Birmann for rails in tunnels had also been found very suitable in Britain, but hot-galvanized coatings had corroded more rapidly than steel in tunnels.

The relation that annual rail wear increased with the square root of the annual number of axles had been obtained from measurements on straight or slightly-curved track, and it was possible that a linear relation applied on more sharply-curved track where mechanical wear was more severe.

Mr Loach's proposal differed only in detail from that which Mr Cookson had mentioned in the discussion and the Author had, in effect, dealt with it when replying to the discussion.

The Author was pleased to find that the values of wear quoted by Mr Herlihy for rails in Ireland were similar to those found for similar circumstances in Britain, and he was grateful for that information.

Correspondence on the foregoing Paper is now closed.—Sec. I.C.E.

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ROAD ENGINEERING DIVISION MEETING

20 May, 1954

Mr Arthur Floyd, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Road Paper No. 45

"The Road Surface as a Factor in Street Lighting" *

by

Alexander William Christie, M.A., B.Sc.

SYNOPSIS

A pedestrian on a lighted street is usually revealed as a dark silhouette against a background formed by the surface of the carriageway. Thus the light from the lanterns is being used to brighten the road surface rather than the objects on it. In the Paper the theory of vision in lighted streets is examined in simple terms in order to show that the effectiveness of the lighting is determined to a large extent by the reflecting properties of the surface.

Each lantern in a lighting system brightens only a limited area of the surface called the "patch." The art of street lighting consists largely in designing lanterns with a suitable light distribution and siting them so that the patches produced are seen to cover the whole of the carriageway from any view-point on it, the aim being to make the surface as bright and uniform as possible whilst avoiding excessive glare.

To the road user each patch appears roughly T-shaped with a head stretching across the road near the lantern and a long tail extending towards the observer. Light is most strongly reflected where it is incident most obliquely, that is, in the tail of the patch—so the surface may be brightened to the greatest extent by directing the light up and down the road rather than close to the lantern. The more the beam is raised so as to shine farther down the road, the greater is the tendency to cause glare, but if the brightness of the road surface increases sufficiently, the glare will not become severe. Since the reflexions from various surfaces can be very different, especially at oblique incidence, the angle to which the beams can be safely raised depends on the surface. The three main types of lantern light distribution are described and their employment is discussed.

The bright patches on a number of dry surfaces have been studied. The results are of value in estimating the effectiveness of lighting installations and in improving their design. They also show how some of the methods used to improve the non-skid properties of roads have reduced reflexion in the tail of the patch, and, therefore, the effectiveness of some types of lantern. An attempt is made to relate the shape of these patches to surface form and lightness of colour. The effect of surface texture on the reflexion of light from wet surfaces is examined.

Introduction

Although street lighting is such a familiar and necessary feature of urban life, it is one which is not well understood by the great majority of road

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users. In particular, it is seldom appreciated just how much the effectiveness of the lighting depends on the nature of the road surface. It is the purpose of the Paper to examine this dependence in some detail and to discuss its bearing on street-lighting practice and road-construction methods.

Basic Ideas

Silhouette Vision

One of the most important and more difficult of the many functions of street lighting is to reveal to road users the presence of objects on the carriageway. It is particularly important that drivers should easily detect pedestrians or cyclists.

An object is visible to an attentive observer if there is sufficient contrast of brightness or colour between the object and its background, or between different parts of the object. The brightness levels encountered under street lighting are much lower than those prevailing during daylight. At low brightness levels the eye loses much of its sensitivity to differences of colour and its ability to resolve small details. Thus in lighted streets, the visibility of objects depends mainly on the existence of a brightness contrast between the object as a whole and its background. Silhouette vision occurs when the object is the darker, and reversed silhouette version when the object is the brighter (see *Fig. 1*, facing p. 512).

It might be expected that street lanterns would make the objects bright, as do vehicle headlamps, but in fact the road surface is usually brightened more than the objects so that the dark silhouette is more usual. Only occasionally is the silhouette reversed, for instance in the case of a pedestrian in light-coloured clothing standing just behind a lantern and receiving the maximum amount of light on the side facing the observer.

Each lantern in a lighting system brightens only a limited area of the surface called the " bright patch " or simply the " patch " (see *Fig. 2*). The art of street lighting consists mainly in designing lanterns with a suitable light distribution, and siting them so that, viewed from any point on the carriageway, the patches produced cover the whole carriageway. The aim is to make the surface as bright and uniform as possible, whilst avoiding excessive glare (see *Fig. 3*, between pp. 512 and 513).

In order to discuss these ideas more precisely, and in greater detail, it is necessary to introduce and explain some of the terms used by the lighting engineer.

Lighting Terms

Definitions are given below for the more important photometric terms. The aim has been to make the definitions simple rather than precise. The usual units and abbreviations used in Great Britain are included.

The " intensity " of a light source in a given direction is the rate at which it radiates light in that direction. The unit is the " candela."

Formerly the unit was the candle. The intensity is sometimes loosely called the candle-power.

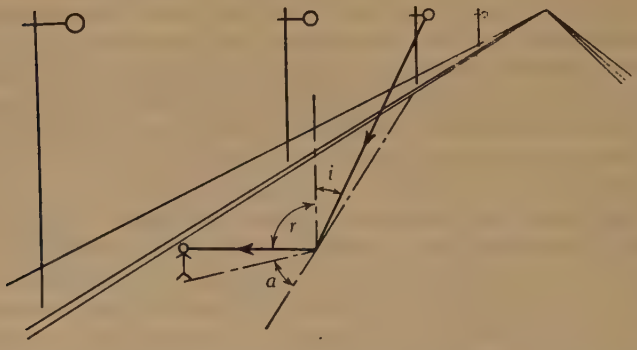
A " uniform light source " is a simple hypothetical source giving the same intensity in all directions. In practice, a light source has different intensities in different directions.

The " illumination " on a surface is the rate at which light falls on unit area of the surface. The unit is the " lumen per square foot."

The term " luminance " is now used (instead of the more familiar term brightness) for the rate at which light is emitted by, or reflected from, unit area of a surface in a given direction. The unit is the foot-lambert.

The " luminance factor " of a surface is the factor by which the illumination (in lumens per square foot) must be multiplied, to give the luminance

Fig. 4



THE GEOMETRY OF REFLEXION

(*i* denotes angle of incidence ; *r* the angle of reflexion or view ; and *a* the azimuth. The luminance factor depends upon *i*, *r*, and *a*.)

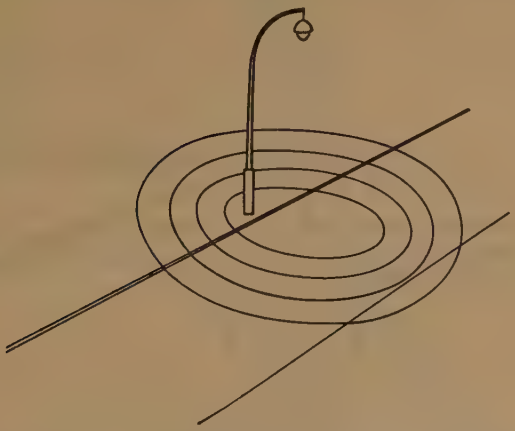
(in foot-lamberts) which it produces. A simple hypothetical type of surface is the " uniform diffuser," which has a constant luminance factor quite independent of the directions of incidence and reflexion. Many matt surfaces approximate to this except for glancing angles of incidence and reflexion. The luminance factor of a uniform diffuser cannot exceed 1·00—in the case of a light surface it may be nearly 1·00 ; for a very dark surface it may be about 0·01. Most surfaces are shiny to some extent and are therefore " preferential diffusers," with luminance factors which vary with the angles of incidence *i*, reflexion *r*, and azimuth *a*, (see *Fig. 4*). The luminance factor of a preferential diffuser for some conditions may be very much greater than one and much less than one for other conditions. As surfaces become more polished they tend to become " specular or mirror reflectors," which, for a given direction of incidence, have a luminance factor of zero in every direction except in the direction of mirror reflexion

($a = 0, r = i$ in *Fig. 4*). Thus, with a single-point source, a mirror surface appears bright only at one point, namely the mirror point where the image of the source appears.

Reflexion of Light by Road Surfaces and Pedestrians' Clothing

Road surfaces are preferential diffusers, a fact which may be demonstrated as follows. If a street lantern which emits a uniform intensity were mounted over a road surface which was a uniform diffuser, then the contours of equal illumination on the surface would be concentric circles with

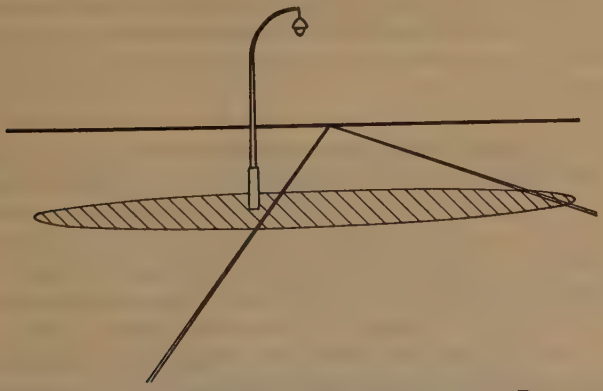
Fig. 5



Circular Contours of Equal Illumination on the Surface from a Uniform Source

(If the surface is a uniform diffuser, these circles are also contours of equal luminance, that is to say, the patch is circular.)

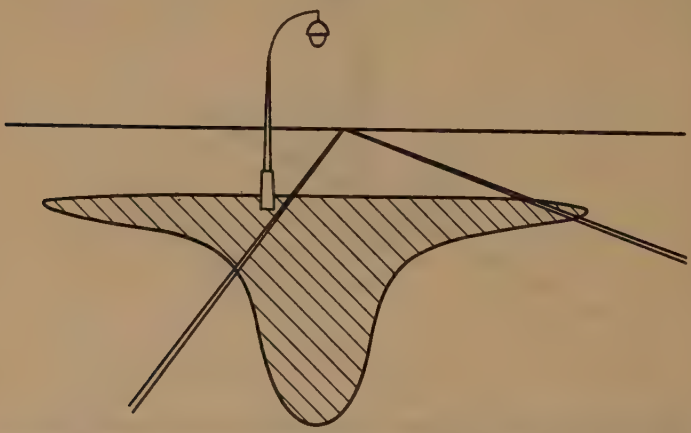
Fig. 6



Circular Patch Produced by Uniform Source on Uniform Diffuser Appears as Narrow Ellipse to Road User

their centres under the lantern (see *Fig. 5*). Since the luminance factor of the surface would be constant, these circles would also be contours of equal luminance so that the bright patch on the surface would be circular. To a road user with a very oblique view of the surface, the patch would appear as a narrow ellipse stretching across the road (see *Fig. 6*). However, the patch obtained on a typical road surface is roughly T-shaped, comprising a wide head stretching across the road but reaching only a short distance behind the lantern, and a long tail extending towards the observer (see *Fig. 7*). This shows that the reflexion is preferential because just as much

Fig. 7



T-Shaped Bright Patch Produced on Typical Road Surface by Uniform Light Source

light is emitted behind the lantern as in front of it. Under the lantern the reflected light is diffused, and the luminance factor depends on the colour and lightness of the surface, and is of the order of 0·1 ; in the tail of the patch there is strong preferential reflexion and the luminance factor rises to about 100.

It is clear that the road surface is most effectively brightened if, instead of having a uniform intensity, street lanterns direct much of their light into two beams, one up and one down the road. By taking advantage of the high luminance factor in the tail of the patch the general surface luminance can be made higher than if the light were emitted downwards into the head of the patch, where the luminance factor is low.

Pedestrians' clothing, on the other hand, behaves rather like a uniform diffuser at the angles applicable to the street-lighting problem ; in the majority of cases the luminance factor is less than 0·1. These facts largely explain why the dark silhouette is the commoner case in lighted streets.

Whilst the luminance factor of road surfaces is greatest for obliquely

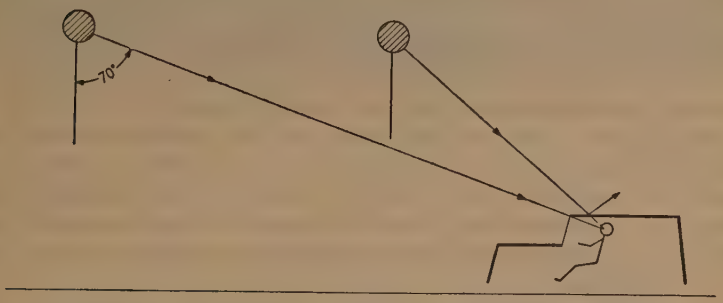
incident light, the extent to which light may be beamed out to take advantage of this is restricted by the tendency to cause glare to road users.

Glare

It is customary to distinguish two main types or effects of glare, disability glare and discomfort glare.

Exactly how disability glare arises is not certain, but its mode of operation has been likened to the production of a veiling luminance or luminous fog over the whole visual field which effectively reduces the contrast between object and background, and therefore reduces the visibility.

Fig. 8



LIGHT EMITTED BY STREET LANTERNS AT LESS THAN ABOUT 70° FROM DOWNWARD VERTICAL CANNOT CAUSE GLARE

(Drivers eyes usually shielded by roof of vehicle.)

Discomfort glare is the term used to describe the sensations of distraction and annoyance which are experienced when glare sources are present in the field of view.

Different factors affect the severity of the two types of glare to varying extents and a full discussion of the problem would involve the use of mathematical formulae. The following discussion is over-simplified and ignores these differences, but it should serve to explain the general nature of the problem.

The effect of a glare source decreases the farther it is removed from the line of sight. For this reason low lighting columns are not employed on traffic routes, the mounting height of the lanterns being standardized at about 25 feet.

The glare experienced by a driver depends, of course, on the intensity directed towards him. When he is near a street lantern, the lantern is screened by the roof of the vehicle, so that intensities emitted at angles of less than about 70 degrees from the downward vertical cannot reach the driver's eyes and so cause glare (see *Fig. 8*). There is therefore no necessity to limit the intensities below this angle. If the intensities at greater angles

are kept low, glare is low ; lanterns in which this is done are called cut-off lanterns.

The glare depends also on the luminance of the rest of the field of view, in particular, on the luminance of the road surface ; it is less severe when the luminance is high than when it is low. Thus, large intensities are permissible above 70 degrees provided that the surface is sufficiently bright. Most non-cut-off lanterns emit their greatest intensities well above 70 degrees in order to form bright patches with long tails. When the luminance factor is sufficiently high in the tail of the patch, glare is not serious. But the greater the angle at which high intensities are allowed, the more numerous are the lanterns which contribute to the glare, and there must be a very rapid rise in surface luminance to offset this. The extent to which it is permissible to raise this angle varies for different surfaces.

The Importance of the Road Surface in Street Lighting

To conclude the discussion of basic ideas, the alternative methods of improving the visibility on a lighted road may be considered in the following hypothetical case. Suppose that, to increase the surface luminance to n times its original value, it were possible either to resurface a road so as to increase the luminance factor n times for each direction of incidence and reflexion, or to increase by n times the intensity of the lanterns in every direction. Then which action would be the more desirable from the point of view of visibility ?

The surface luminance would be the same in both roads, but in one the lanterns would appear n times brighter than in the other. Thus, as regards glare, it would certainly be better to increase the luminance factor of the surface rather than the power of the light sources.

Whilst changing the surface would leave objects on the road as dark as they were before, raising the light output of the lanterns would brighten the objects as well as the road. Since silhouette vision is the commoner case it is clear that greater contrast, and therefore better visibility, would be achieved by improving the surface.

A photograph of a road on which visibility has been decreased and glare increased by a change of surface is shown in *Fig. 9*. The old surface, a smooth light concrete, has been partly surface-dressed using an aggregate of dark crushed stone.

Types of Street Lighting

The design of street-lighting installations in Great Britain has been guided by the reports of the Departmental Committee on Street Lighting, whose final report[1] published in 1937, recommended that roads requiring lighting should be divided into two groups, as follows :—

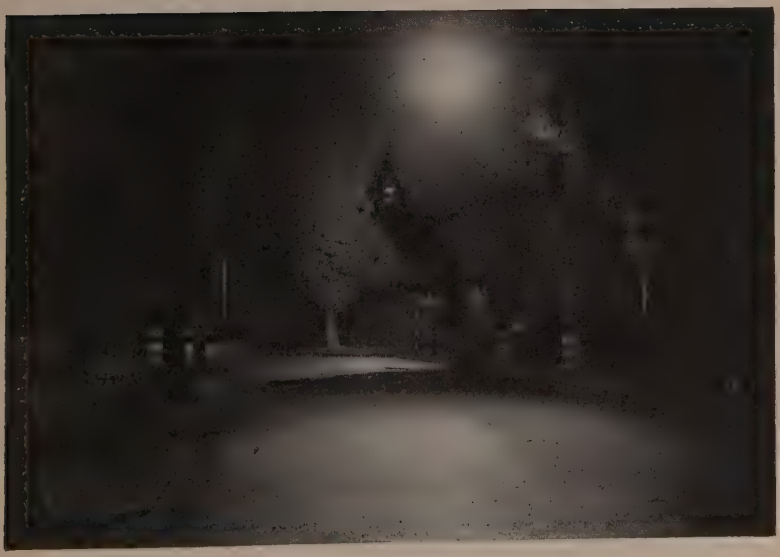
Group "A"—Traffic Routes. All those roads that form the main

[1] The references are given on p. 521.

Fig. 1

Vision by Reversed Silhouette and Silhouette

Fig. 2



The Bright Patch Formed by an Individual Lantern

Fig. 3

Overlapping the Bright Patches from Many Lanterns

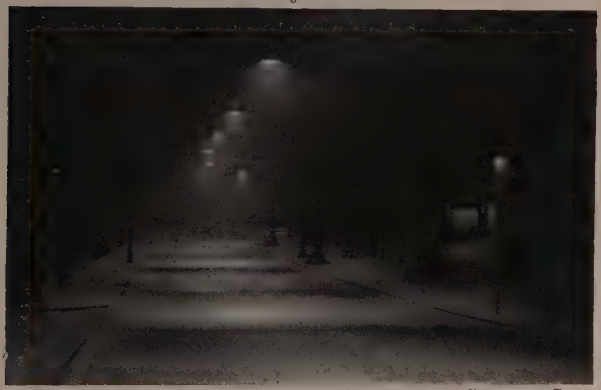
Fig. 9

The Adverse Effect of Dark Surface Dressing

Fig. 11

The Bright Patch Formed by a Cut-off Lantern

Fig. 12



Complete Installation of Centrally Suspended Cut-off Lanterns

Fig. 14

Transitional Lanterns Intermediate Between High- and Medium-Angle Beam Types

Fig. 17

WET ROAD WITH FINE SURFACE TEXTURE

Fig. 18

WET ROAD WITH COARSE SURFACE TEXTURE

approaches to or traverse important centres of population, or pass through detached built-up areas, and on which there is appreciable pedestrian traffic.

Group "B"—Other roads requiring lighting. Residential roads, etc.

For group "A" installations, a lantern height of 25 feet is recommended and a sufficient light output to render the use of headlamps unnecessary. For group "B" installations, lanterns with a lower light output are recommended mounted at a height of about 15 feet. Group "B" installations are helpful to pedestrians and indicate to drivers the direction of the road, although headlamps are generally required to ensure that obstacles on the carriageway are seen. Part I of a British Standard Code of Practice for Street Lighting[2] was issued in 1952. This deals with group "A" installations and is based on the recommendations of the Departmental Committee. Part II, which is to deal with group "B" installations, is in course of preparation.

The present Paper deals with group "A" lighting, which is intended to render the use of headlights unnecessary, but much of what is written applies to group "B" lighting also.

Many different types of lantern are used in group "A" lighting and these could be classified in a number of different ways. The ordinary road user would undoubtedly classify lanterns according to the colour of the light which they emit, that is to say, according to the type of light source employed. But it has not yet been convincingly demonstrated that any one colour of light produces better visibility than the others. Financially, one type of light source may be more attractive than another but no attempt is made to deal with the economic aspect in this Paper. Lanterns might be classified according to the optical systems employed to distribute light in the manner required, but similar effects can be produced by different means. In this Paper, lanterns are classified according to their light distribution, since it is probably the most important quality so far as visibility is concerned.

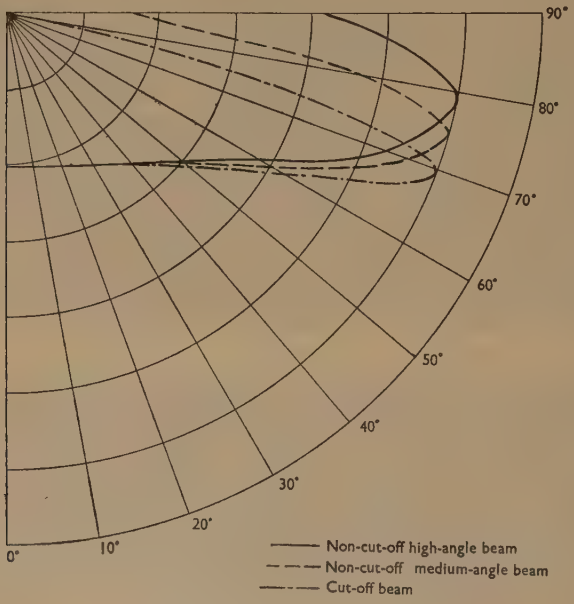
Most street lanterns emit two main beams, one up and one down the road. Thus, the maximum intensities are emitted in vertical planes parallel, or nearly parallel, to the direction of the road. The polar distribution curves in these planes through the maximum intensity are shown (see *Fig. 10*) for the three main types of light distribution—cut-off, non-cut-off medium-angle beam, and non-cut-off high-angle beam. The angle is measured from the downward vertical.

At one end of the range of lanterns is the cut-off lantern which emits little light above 80 degrees. Often there is a screen which cuts off the light completely at the horizontal. Usually the peak intensity is near 70 degrees, with a rapid fall in intensity above that angle. The roof of a vehicle screens its driver from the greatest intensities so that glare may be considered to be negligible. But with lanterns mounted at the usual height of 25 feet, the bright patch on the road surface is only about 90 feet

long (see *Fig. 11*) ; therefore, a lantern must be erected every 90 to 100 feet along the road. Sometimes the lanterns are suspended over the centre of the road, the patches combining as is shown in *Fig. 12*, or the lanterns are mounted at the side of the road in staggered formation (successive lanterns on opposite sides of the road).

At the other end of the range is the high-angle beam lantern with a peak intensity at about 80 degrees, an intensity exceeding half the peak value at

Fig. 10

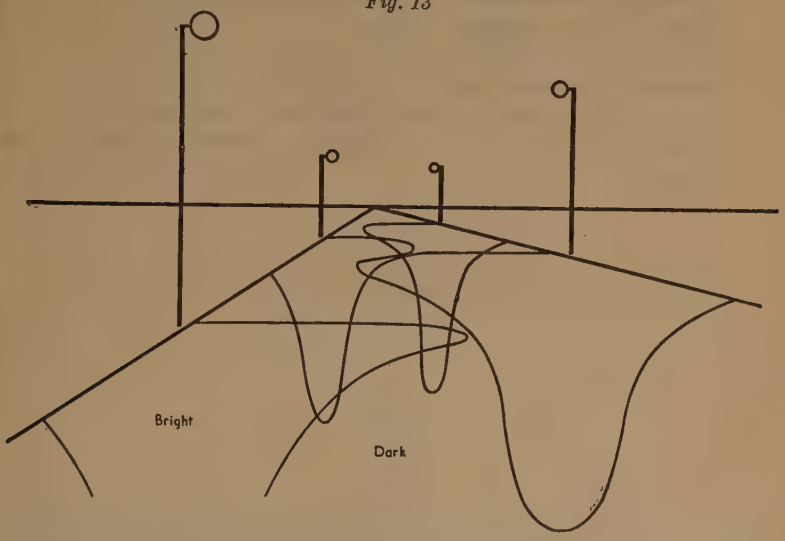


Typical Light Distributions
(Polar curves of intensity in vertical planes through the maximum intensity.)

86 degrees, and an appreciable intensity right up to the horizontal. Long bright patches are produced on a suitable surface (see *Fig. 2*) and long spacing of 120 to 150 feet may be employed. Central suspension is not recommended ; usually a staggered arrangement is employed. The longer spacing is possible because the patches from several lanterns assist in brightening each span (length of carriage between successive lanterns) as is shown in *Figs 3* and *13*. With such a light distribution a certain amount of glare is inevitable because several lanterns emit a high intensity towards the eyes of a road user. It appears that only in the case of surfaces on which very long patches are produced is the general surface luminance high enough to offset these intensities and to keep glare low. In the case of surfaces on which the patches are short, glare may be severe.

Between these extremes is the medium-angle beam lantern with a peak intensity at about 75 degrees. At 81 degrees the intensity is greater than half the peak value ; at 86 degrees the intensity is less than half the peak value. This type of lantern may be employed either in staggered or central arrangements at a spacing of about 120 feet. Medium-angle beam

Fig. 13



PERSPECTIVE DIAGRAM OF BRIGHT PATCHES ON ROAD LIGHTED BY NON-CUT-OFF HIGH-ANGLE BEAM LANTERNS

lanterns or transitional types between the high- and medium-angle beam types appear to be most popular at the present time (see *Fig. 14*).

INVESTIGATIONS OF ROAD SURFACE REFLEXION CHARACTERISTICS

Handling Reflexion Characteristics

Since the luminance factor of a road surface is a function of three variables (*i*, *r*, and *a* in *Fig. 4*, p. 508), a large mass of data is required if its value is to be accurately known over the whole range of conditions encountered in street-lighting problems. A simplified method has, therefore, been developed at the Road Research Laboratory which permits a close approximation to the luminance factor to be derived from a selected set of values, which may be presented in the form of a single chart. These approximate values appear to be sufficiently accurate for most practical applications.

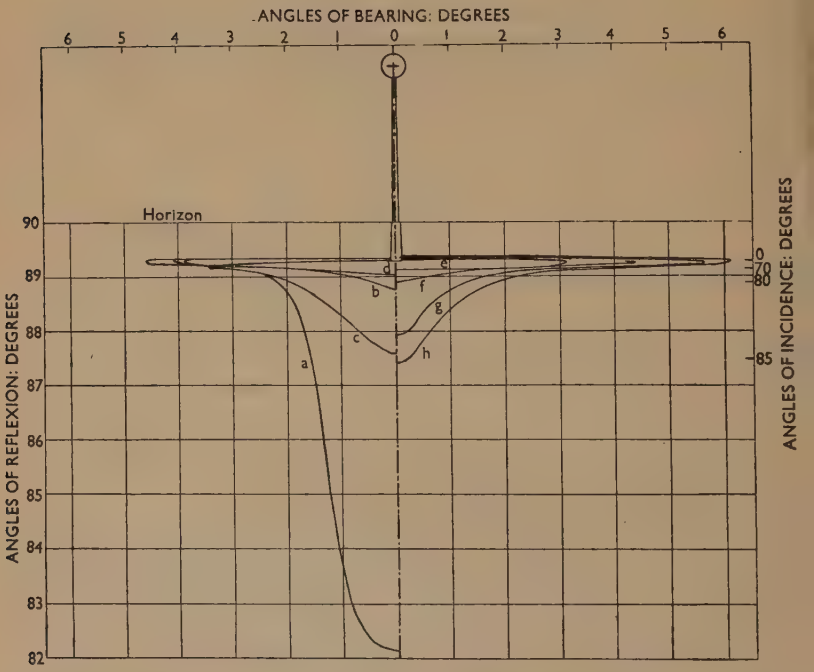
Whilst such charts are sufficient for research and design purposes, they do not present an immediate picture of how the surface is brightened by a street lantern. It is useful, therefore, to derive from the charts a series of

contours of equal luminance to show the shape of the patch formed by a standard light source under a standard set of conditions. The conditions selected for the standard patch are listed in Table 1.

TABLE 1.—CONDITIONS FOR THE STANDARD PATCH

Lantern light distribution : uniform intensity of 1,000 candelas.
Lantern mounting height : 25 feet.
Height of observer's eyes : 5 feet.
Distance of observer from lighting column : 400 feet.
Correct distance from which a perspective drawing (see *Fig. 15*) should be viewed : 16 times height of lantern on scale of the drawing.

Fig. 15



a : Compressed rock asphalt.
b : Non-skid rock asphalt.
c : Rolled asphalt with precoated chippings.
d : Bituminous sand carpet.
e : Surface dressing of fairly light quartzite.
f : Surface dressing of dark hornfels.
g : Surface dressing of light flint gravel.
h : Machine-finished concrete.

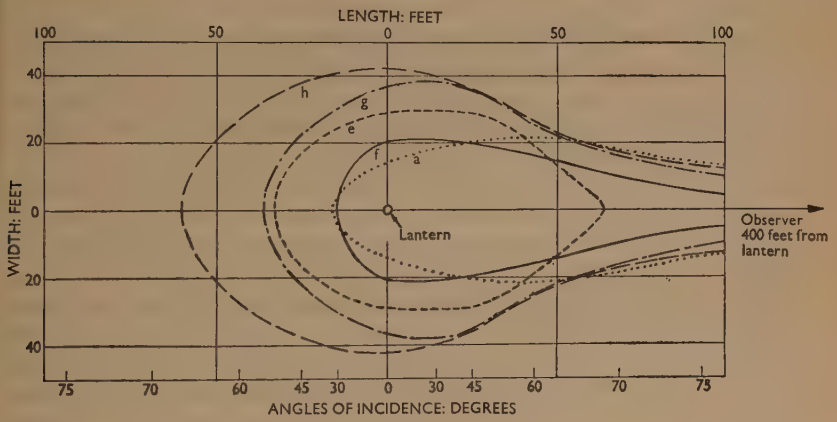
PERSPECTIVE DRAWING OF 0·04 FOOT-L. CONTOUR FOR STANDARD PATCHES FOR DRY SURFACES (SEE TABLE 1)

For the lantern a uniform light source of 1,000 candelas is used. Although street lanterns never have a uniform intensity it is easy to see from a complete set of contours how the standard patch would be modified by the substitution of a lantern with any given light distribution.

In order to facilitate a comparison of the shapes of the standard patches obtained on the dry surfaces considered in the next section, only a single contour is given for each—that corresponding to 0·04 foot-lambert (see *Figs 15* and *16*).

A patch has, of course, no sharp edge. The factors that determine the

Fig. 16



LEGEND

. . . . . a—Compressed rock asphalt.
– – – – – – – e—Surface dressing, fairly light quartzite.
———— f—Surface dressing, dark hornfels.
— . — . — g—Surface dressing, light flint gravel.
— — — — h—Machine-finished concrete.

PLAN VIEW OF 0·04 FOOT-L. CONTOUR OF STANDARD PATCHES FOR DRY SURFACES (SEE TABLE 1)

position of the apparent edge are probably quite complicated and it is not suggested that the chosen contour would correspond exactly with visual estimates. Whilst this contour has been chosen somewhat arbitrarily, the shape and size of contours corresponding to a definite luminance are good criteria of the relative effectiveness of various surfaces in different types of lighting.

A driver on the road has a very oblique view of the road surface. To show the standard patch as he would see it, a perspective drawing, similar to *Fig. 15*, must be made. For some purposes (for instance, to examine the head of the patch) a plan view (see *Fig. 16*) may be more appropriate.

These are still the contours for the standard observer on the road; an observer in an aeroplane would see quite a different type of patch.

Some Results for Dry Surfaces

A perspective view of the 0·04 foot-lambert standard contours for a number of dry surfaces is given for comparison in *Fig. 15*.

Curve a is for a smooth compressed rock asphalt laid in 1929 and still in existence on a London street. The modern counterpart of this surface is the " non-skid rock asphalt " for which the standard patch is shown by curve b. The introduction of the protruding stone aggregate to improve the non-skid properties of the surface has reduced severely the length of the patch.

Short patches are obtained not only on coarse surfaces but also on roads with a fine surface texture formed by protruding aggregate particles of small dimensions (see, for example, the standard patch for a bituminous sand carpet shown in *Fig. 15* by curve d). Within limits it is the shape of the surface, and not the size of its features, which is important.

In the case of rolled asphalts with precoated chippings, the chippings are generally rather sparsely distributed and protrude only slightly, so that the reflexion properties depend on both the chippings and the mortar. In 1934 Waldram[3] published data for a rolled asphalt with precoated chippings (B.S.342, 1928) which indicate a standard patch much longer than that given as curve c in *Fig. 15*, which is for a recent example (B.S.594, 1945). Although the type of chippings employed is not the same, much of the difference is probably attributable to a difference in the mortar. The older surface probably had a higher bitumen content than would now be used.

It was the existence of surfaces such as the compressed rock asphalt and the older rolled asphalt with precoated chippings, discussed above, that encouraged the use of high-angle beam lanterns. Since a long spacing could be used between the lanterns, good results could be obtained at relatively low cost. Many of the older lighting installations now appear glaring, because the patches are much smaller and the general surface luminance much less than was the case when the installations were erected. In many of the newer installations medium-angle beam lanterns are employed.

One reason why the shape of the surface features is important is that it determines the extent to which one element of the surface shades others from illumination or view. On close inspection, a surface is seen to consist of a mosaic of dark and light elements; whilst some of the elements are reflecting light, others are shaded from the illumination and lie in shadow. The dark areas are most prominent in the tails of the patches on surfaces with projections. Thus, projections may greatly reduce the reflexion of obliquely incident light and prevent the formation of long patches. Even a crude classification of surfaces according to their shape would be helpful.

It is, therefore, proposed to refer to surfaces with projecting elements as " peaky," and to use the adjective " even," to denote the absence of these projections.

The most obviously peaky surfaces are new surface dressings of crushed stone, for not only do the aggregate particles stick out a long way from the binder but the particles themselves have a peaky surface imparted by the crushing process. An example of a surface dressing of fairly light quartzite is shown in *Fig. 15* by curve e. This surface has had about a year's wear. The patch is very short. Sometimes stones polish under the wheels of traffic and a lengthening of the patch results. The standard patch for a surface dressing with dark aggregate of the hornfels trade group which had begun to polish in this way is shown in *Fig. 15* by curve f.

It might be expected that a much larger patch would be obtained on a surface dressing using an aggregate with a high proportion of smooth rounded particles. This is borne out in the case of a light flint gravel surface dressing for which the standard patch is given in *Fig. 15* by curve g.

It appears that sharp projections are necessary to prevent skidding when the road is wet. Thus, it is important to consider whether the surface texture required to prevent skidding also prevents the formation of long patches by street lanterns when the surface is dry. The available experimental evidence suggests that it tends to do so. The only really long standard patch shown in *Fig. 15* is that for the compressed rock asphalt (see curve a). The bituminous sand carpet, which has a very high resistance to skidding, has a very short standard patch. With the surface dressings, the patches are longest when the aggregate is either naturally smooth or when it has been polished by traffic.

Peaky surfaces are more diffusing than even surfaces and the head of the patch is more pronounced, but its actual size is dependent on the lightness of colour of the surface. *Fig. 16* shows in plan the 0·04 foot-lambert contours for some of the surfaces. For the very even compressed rock asphalt the contour extends only a short distance behind the lantern, although the surface is fairly light. The contour encloses a larger area behind the lantern in the case of the peaky surface dressing of dark hornfels, although it is darker. The patch for the surface dressing of fairly light quartzite extends considerably farther. The light flint gravel gives an even larger patch.

Of the surfaces studied, a machine-finished concrete gives the patch with the biggest head. The plan view is shown in *Fig. 16* and the perspective view in *Fig. 15*, by curve h.

Thus, although peaky surfaces do not give long patches, they give patches with large heads provided they are light in colour. Such surfaces are ideal for cut-off lighting. Dark peaky surfaces are difficult to light satisfactorily, but if cut-off lighting is employed glare may be kept down.

A disadvantage of cut-off lighting is the close spacing that is necessary ; this increases both capital and running costs. Cut-off lighting is not widely

used in Great Britain although it is popular on the Continent, where people appear to be less tolerant of glare. Amongst ordinary road users in Britain there appears to be a preference for cut-off lighting. There is at present no evidence that the use of cut-off lighting is increasing in Britain ; it may even be decreasing, since the ban on the use of non-cut-off lighting in the vicinity of airfields has been relaxed. However, there is a move away from high-angle lighting towards medium-angle lighting. It is to be hoped that this tendency will continue (unless, of course, road surfaces again become really suitable for high-angle lighting).

The new data support what is found in practice, namely, that lightness of colour is a desirable quality of the surface of a lighted street, particularly in the case of surfaces which tend to reflect diffusely (for example, surface dressings). From the point of view of street lighting, concrete appears to be a very suitable surface and the extension of its use in urban streets would assist visibility.

Wet Weather Reflexion

When rain falls, the road surface may become flooded if the surface texture is fine. The bright patches then degenerate into very narrow bright streaks, as shown in *Fig. 17*, facing p. 513. On the more rugged types of surface this stage is rarely reached. Usually the patches remain quite wide (see *Fig. 18*). Whether or not a surface will flood is closely related to the " texture depth " of the surface.

Texture depth is measured by pouring a known volume of fine sand on to the surface and spreading it into a circular patch so that the valleys are filled in to the level of the peaks. The volume of sand divided by the area of the sand patch is the texture depth. It is in fact the average thickness of the sand layer.

By comparing the widths of the patches on a number of bituminous surfaces of different texture depth, it has been found that the minimum texture depth required to give patches of reasonable width during light rain was about 0·013 inch. This value is rather greater than that obtained on most rolled asphalts with precoated chippings but is much less than that obtained on normal surface dressings with $\frac{3}{8}$-inch aggregate. Some carpets with $\frac{3}{8}$-inch aggregate have a texture depth of about 0·013 inch.

The conditions shown in *Fig. 17* are dangerous because it is very difficult to see objects in the dark spaces between the streaks and along the kerb's edge. If the texture depth is not allowed to fall below 0·013 inch such conditions should not arise except in fairly heavy rain.

Conclusion

The treatment of the theory and practice of street lighting given in this Paper is necessarily very incomplete. Those seeking fuller information would do well to study Waldram's textbook.[4] It is hoped, however, that

the Paper will contribute to a more general understanding of the importance of the road surface in street lighting, and will assist engineers to choose appropriate lighting systems and surfacings.

Acknowledgements

The Author wishes to place on record his indebtedness to his colleagues at the Road Research Laboratory.

The work described in this Paper was carried out as part of the programme of the Road Research Board of the Department of Scientific and Industrial Research. The Paper is published by permission of the Director of Road Research.

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The Paper is accompanied by nine photographs and eight sheets of drawings from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

Mr John Carr, remarking that the subject was a really difficult and complex one, said he proposed to deal with the relationship between the highway engineer and the road lighting engineer. In some cases they were one and the same person, who often had to make a difficult decision between the claims of road surface on the one hand and lighting on the other. The road engineer was concerned with getting a good road surface which was economic to build and which had good non-skid qualities, whilst the lighting engineer wanted a road which would give him a means of erecting an economic lighting installation; as the Author had indicated, the two requirements did not always agree. The difficulty was, of course, that from a non-skid point of view the rougher the surface the better, not necessarily because it was more non-skid but because it appeared to be so, and, therefore, gave drivers greater confidence, enabling them to deal with emergencies much better than if they thought that they were on a

slippery surface. That was a difficulty which had to be resolved between the two sides.

It had to be remembered, of course, that when a new surface was laid down, unless it was concrete or had a gravel surface dressing, it was usually composed of an aggregate and some kind of binder, generally tar or asphalt, which were dark. The point was often overlooked that the dark binder was only a small percentage of the whole, perhaps between 6 and 9 per cent, so that in the course of time the very dark appearance of a new road surface would gradually disappear, and there would remain the colour of the natural aggregate, always, of course, tempered and interspersed by the fragments of binder which showed through the mosaic appearance of the road. It was important, therefore, to get as light a coloured aggregate as was possible, economics permitting.

A suggestion which he had made to the research workers, and which might appear rather fantastic, was that if it was going to be very difficult to get a light road surface as such, the possibilities of whitening it should be explored. After all, tar was blown on to it as part of the process of surface dressing, and it might be feasible to blow some kind of white mist spray on to a newly-surfaced road to give it a white appearance instead of the black appearance which it would ordinarily have. How long that would last and how much it would cost would be the factors which dictated whether or not it would ever be any use.

Weather conditions were another problem which confronted both parties. As the Author had indicated, the weather played a big part in dictating the shape of the patch of light seen by the driver, and the rugosity and other characteristics of the road also had their effect, so that when the road engineer and the lighting engineer were the same person, he found it difficult to decide which was the more important.

Fig. 8 was of great interest, because it illustrated an experience which was commonly recognized but not always understood. Glare was an exceedingly difficult subject and, although possibly there had been a great deal of research on the subject, there was a need for a much clearer exposition of the relevant facts. It seemed that glare could be very annoying and distracting in certain circumstances and not in others. If there was a great amount of light falling on the surface the glare effect would not be so great, whereas if there was little light reflected from the road then, with only a moderate amount of light coming directly into a driver's eyes from a high-angle lantern, the glare might be very severe.

The conditions at dawn and dusk were very difficult, and again it seemed to Mr Carr that a light-coloured road surface would be helpful in every respect.

Dr A. R. Lee said that the Paper showed clearly what were the basic principles which had to be considered when the engineer had to make a decision on matters affecting the lighting of his roads. Dr Lee had been

nterested to hear Mr Carr say that the engineer might in fact be two engineers, one responsible for the roads and the other for the lighting of them. That certainly made matters a little complicated and close co-operation was clearly required if the best results were to be obtained. The Author had given the first results of the first stages of his investigations; he had a great deal of work still in hand, the results of which were eagerly awaited.

There was no doubt, from the photographs contained in the Paper, how large could be the contribution made by the type of road surface to visibility. Presumably with a given lighting system the engineer could choose the surfacing which would give the best results. However, in built-up areas, in order that there should be the least interference with traffic, the engineer probably liked to have a road surface which required the minimum of maintenance and at the same time was of a durable non-skid material of pleasing appearance. When constructing a new road, therefore, he might be faced with the problem of what was the most economic combination of the type of durable road surface and the type of street lighting, and whether it was worth while economizing on his lighting installation by the choice of a particular type of surfacing material. Had the Author thought of that aspect of the question? Alternatively, was it better to choose some arbitrary lighting scheme and then select the most economic road surfacing to suit it?

The second alternative raised the question of what was the best surfacing to use to get the best value from the lighting system. The Author had given a general indication of how that could be arrived at, and the two factors which determined the visibility were surface texture and colour. There was no doubt that *Fig. 9*, which contrasted the effect of concrete and of a dark surface dressing, illustrated very well the enormous changes in the luminance of the road surface which could result from a change in the type of road surfacing material. It would be interesting if the Author could give some further details on the question of texture and colour. In *Fig. 9* the two surfaces were a smooth light concrete and a rough dark surface dressing. Dr Lee's interpretation of what the Author had said was that colour was by far the more important factor when considering non-skid surfacings having a .texture depth greater than the minimum value which was required to avoid specular reflexion in wet weather, and he would like to know whether that interpretation was correct.

That raised the question of which was the more important condition of the road to consider, the dry or the wet condition. *Fig. 15*, showing the distribution of light as seen by the observer, indicated that for a dry road the best result was given by a compressed rock asphalt, but when that road was wet it gave the sort of result shown in *Fig. 17*, and Dr Lee could not imagine a worse condition for the driver of any motor vehicle than that shown there. He thought, therefore, that it was necessary to decide

what emphasis should be placed on the wet and the dry condition, and he suggested that it was the wet condition which should receive the major consideration. Presumably on that basis there was the minimum texture depth required for avoiding specular reflexion in wet weather, and he would suggest that, for surfacings having a texture depth greater than that minimum value, colour was the predominating factor.

It was very important to obtain further information which would help to distinguish between the effects of texture and colour, and he knew that the Author was carrying out experiments on various roads for that purpose. It might be of interest to refer to one which was being made in co-operation with the Slough Borough Council and the Limmer and Trinidad Lake Asphalt Company, to show the type of experiment which was in hand. Two series of surfacings had been laid. One consisted of medium-textured carpets containing 25 per cent of sand and made with various coarse aggregates of different light colours from white to greyish pink ; the other sections consisted of finer textured asphalts, one of which had light coloured chippings which had been coloured, as the previous speaker had suggested, with a white asphalt. Perhaps the Author would say whether he had yet been able to draw any conclusions from that experiment.

That experiment was only one of quite a number which Dr Lee was sure the Author would be doing before he could arrive at definite conclusions on the matter. Unfortunately, such experiments were not easy to carry out. There were restrictions and site limitations both on suitable surfacings and on types of lighting, and a long time was required before definite results could be obtained. It would be interesting to know if model laboratory experiments would provide a useful tool for helping in the solution of the problem.

Mr C. Harper said that the Paper had the special merit of being simple in addition to being informative.

The Author had referred to the B.S.I. Committee which had recently drawn up a Code of Practice. Mr Harper had had the privilege of serving on both the drafting committee and on the main committee and that was another reason for his being particularly appreciative of the Paper. There were other organizations which specialized on the subject ; but he recalled that the Charter of the Institution referred to using the sources of power in nature for the " convenience " of man. It was good to get away sometimes from strict economics and from what might be called the industrial side of the lighting question and think of a " convenient " journey by road. He liked to be able to take a car journey in the evening and to arrive at his destination without having his shirt-front or his temper ruffled by the effects of sudden braking.

On p. 513 the Author had stated: "But it has not yet been convincingly demonstrated that any one colour of light produces better visibility than the others." Mr Harper was pleased to see that statement in the Paper, because it indicated a good line of further research. He had recently been

esponsible for recommending a certain type of lighting on a big estate here there would be 7,000 people, with roads to correspond, and he had ound great difficulty in answering when asked if there was a real visibility reference between mercury and sodium. Many people, even engineers, till looked upon the question of how colour affected visibility as a modern ne, arising solely out of " mercury *versus* sodium," but it was in fact nearly 00 years old and was keenly debated during the time that lighthouse lumination was passing from candles and oil to the electric arc. Most eafarers and lighthouse keepers were quite convinced that the yellow low was better in mist and fog. It was a great pity that it had not yet een decided, because the laymen sitting round a council table were all uite certain that one or the other was easily the better, and it was only he technicians who were still " sitting on the fence."

On p. 519 appeared the statement that " It appears that sharp projec-ions are necessary to prevent skidding when the road is wet," and Mr arr had already shown that stress should be laid on the word " appears." Ir Harper recalled, therefore, the controversy in the early days of railways, hen some people thought that a rack and pinion arrangement would be ecessary to prevent skidding or slipping, yet it had been found satisfactory o have a perfectly smooth wheel on a perfectly smooth rail. He suggested hat even that question had not yet been properly decided.

On p. 520 there was the statement that "Texture depth is measured by ouring a known volume of fine sand on to the surface and spreading it nto a circular patch so that the valleys are filled in to the level of the eaks. The volume of sand divided by the area of the sand patch is the exture depth. It is in fact the average thickness of the sand layer." Ir Harper wondered if that was so. If a series of closely packed regular yramids covered the area and the area was then levelled by filling with and, the volume of sand used was not the base area multiplied by half the eight. Would the Author explain the apparent fallacy ?

The Author's photographs were very good but, with one exception, hey had a defect common to almost all photographs of street lighting, in howing the position as seen from the left—that being the normal driving osition. It was, however, far more informative in a way to show the osition from the right, because a driver was usually most agitated when e was overtaking traffic. Those who drove on the Continent knew the eeling of anxiety which was caused by having to keep to the right, and ow, at first, one felt that one was on the wrong side of the road. That as the sort of agitation to which Mr Harper referred.

He read an extract from a book [5] which members would find informa-ve, because it showed how far they had gone in the approach to street ghting since 1921.

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[5] A. P. Trotter, " Elements of Illuminating Engineering." Pitman, London, 921, p. 92.

> "There are two distinct objects in the lighting of a road or street. One is to guide the traffic by means of lamps which act primarily as beacons. The illumination on the ground is so small that it is inappreciable except in the neighbourhood of the lamps. This beacon lighting constitutes perhaps 80 or 90 per cent of the lighting of the roads and streets of this country."

Another quotation which he thought would also be found interesting, appeared on p. 94 of Trotter's book :—

> "Engineers and surveyors of the local authorities are generally responsible for public lighting, and it becomes their duty to see that street lighting is carried out either by the gas or electricity department of the local authority, or by a company. In 1911, a Joint Committee consisting of gas engineers, electrical engineers, and municipal and county engineers and surveyors discussed these questions at great length. A sub-committee of four municipal engineers examined the lighting of a number of streets in the West End of London and prepared a list of streets, grouping them into first-rate thoroughfares, auxiliary roads, and side or unimportant streets. Another sub-committee of three men who were accustomed to the use of illumination photometers [in other words, the "backroom boys" from the research laboratories] made measurements (about 88 in one night) in some of the same streets, and arranged these in order of minimum illumination. It turned out that the streets thus arranged fell exactly into the same classes as those determined by the judgment of the municipal engineers."

Mr Harper felt that *Fig. 4* in the advance copy of the Paper was misleading. It was difficult at first glance to recognize which lines were in which plane, or whether the drawing was an oblique projection or a perspective. Which lines were construction lines, and which was a physical line—the kerb ? He hoped that there would be a clearer diagram to go in the permanent records of the Institution.*

Mr G. Grime said that he had the privilege of being one of the Author's colleagues, and therefore knew that what the Author had put in the present Paper was really a summary—a skeleton, as it were—of the work which had been going on. He would like to call attention to one point which arose from the Paper and which bore on the question of the visibility of markings on the road. The Author had shown that in order that an object on the road should be visible it must present a contrast to the road surface. Generally speaking, the object was seen in silhouette and was darker than the road surface. The Author had also pointed out that

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* Since the date of the discussion, a clearer drawing has been prepared and is reproduced as *Fig. 4* on p. 508.—Sec. I.C.E.

xcept at the head of the patch the contrast was one of light and shade nd not of colour, so that in order that a road marking should be visible n the road it must also present the same contrast of light and dark.

The second point to which Mr Grime wished to call attention was that he angles at which one viewed objects on the road in street lighting were ery small ones; that was to say, if the angle of reflexion were considered, ot as given in the Paper, but as the angle between the ray of light which ame to one's eyes and the road surface, it was a very small one and was n fact almost grazing incidence for most of the things that one wanted to ee. Whatever was put on the road surface, therefore, had to be visible n those conditions; it had to give a contrast of light and shade at those ery small angles.

The most important sort of road marking with which they had been oncerned—and the Author had had a good deal to do with that—was he marking for pedestrian crossings. It was very important for engineers o realize those simple physical principles so that they could improve their edestrian crossing markings. When the Road Research Laboratory ame to study such matters by looking at models, even at that early stage t became quite clear that it was necessary to produce the desired contrast y means of texture. One of the main points which emerged from the resent Paper was the tremendous importance of texture in producing lark or bright road surfaces. To produce a pedestrian-crossing marking hich would be visible in those circumstances, therefore, it was necessary o have different textures, one for the white marking and another for the lark marking. If the dark marking was to remain dark it must have a ough texture which, as had been shown, would not reflect light very ell at those grazing incidences. He thought that those principles were uite clear.

Then came the question of how to produce that contrast of textures nd maintain it on the road. He was not going to suggest that the roblem had been altogether solved; but, although now there were flashing eacons to show up the zebra crossings at night, he thought that it was till worth while, if it did not mean too much expense, to produce a con- rast between the light and the dark stripes by means of a difference in urface texture, for the reasons which he had given and because that ontrast was also effective in wet weather. In wet weather in daylight was now fairly well known, he thought, that the zebra markings could isappear, but with contrasts of texture the black tended to remain black. Ir Grime illustrated his point by means of two lantern slides. The first owed a marking of the ordinary type, no doubt laid in the early days, ith the same rather shiny surface texture all over the crossing. Only ith very great difficulty was it possible to distinguish the white from the lack. The second slide showed a pedestrian crossing where the black rips had been roughened and did not reflect the light from the street mps. It was probably true to say that the marking was not quite what

it appeared to be by day, because it had become a pattern of black holes in the road, but the distinctive pattern was retained.

The point was, in his view, an important one, and it applied, of course, not only to zebra markings but to all markings, such as white lines in the centre of the road and so on. It was possible to have the white part too smooth. It was possible, for example, to lay down a white line at the centre which was constructed of very smooth material, and from most points of view that would appear black, because there was no reflexion coming direct to the eye. It might flash up at a particular angle, but in general it was better not to have those stripes quite so smooth.

Colonel Sidney Green noted that the Author had carefully avoided controversy on types of light source. In that connexion it was interesting to find Mr Carr taking part in the discussion, since it was in his territory that a notable large-scale experiment had taken place in pre-war days.

On p. 513 the Author had referred to installations which " are helpful to pedestrians and indicate to drivers the direction of the road, although headlamps are generally required to ensure that obstacles on the carriageway are seen."

Emphasizing the word " generally," Colonel Green said that any system of street lighting which left a doubt in the mind of a driver as to whether he should use headlamps or not might well prove a danger to pedestrians *crossing the road* and he hoped that that point was being watched.

Mr G. A. Wood asked whether it had been realized, although it had not been mentioned, that an equal amount of dazzle could occur from street lighting from the reflexion from a wet road as from the direct light source. He had found the reflexion dazzle more troublesome than the direct dazzle, because the direct dazzle was shut off by the roof of the car, whilst the reflexion dazzle was not.

He wondered whether attention had been concentrated too much on lighting in fine weather rather than lighting in wet weather. It was in wet weather that the visibility was the poorest and also that the pedestrian was most uncautious, tending to seek shelter from the rain rather than from the dangerous vehicle. A very great source of accidents was the pedestrian who stepped off the kerb suddenly. Good illumination of the footpath might be considered just as important as good illumination of the carriageway ; in fact, given a choice one might even say that illumination of the footpath was more important, because it was there that the pedestrian lurked before dashing on to the carriageway. It was helpful that the colour of the footpath could be varied much more readily than the colour of the carriageway surface.

With regard to the colour of the lighting, not enough consideration seemed to be given to the surroundings of the road. The fence, wall, shop-front, or whatever it might be alongside the road, was as important a background as the road surface, except straight ahead. The colour of

he light might be important when considered in relation to the colour of he surroundings.

*** **Mr S. M. Lovell** thought that the problem of street lighting as et out in the Paper revealed the fundamentally difficult and conflicting nterests. It was desirable that that difficulty should be more widely nown and a little more co-ordination between the specialist lighting ngineer and the specialist highway engineer would be helpful.

However, since such co-operation appeared difficult to obtain, at least t present, would it help if the problem were treated in the following way ? 'he road surface could be made generally dark in colour or of a material vhich would eventually darken. That condition could be accepted and he edges of the road then made of a contrasting light colour so that ilhouettes were more clearly shown up. Mr Lovell would appreciate the uthor's expert view on that suggestion.

The Author, in reply, referred to Dr Lee's request for his opinion as o whether colour was the most important factor when considering non-kid surfacings having a texture sufficient to avoid narrow streaks in wet veather. If he had simply asked whether colour was the most important actor the Author would say that it was not. But, so far as could be idged at present, the requirements of non-skid performance appeared to mit what could be done by means of a choice of surface texture by elimin-ting the very even surfaces which could be lighted relatively cheaply y high-angle beam lanterns spaced at relatively long intervals. Thus, olour appeared to be the most important quality in the sense that it was ne one most amenable to variation. By choosing light materials in reference to dark materials appreciable improvements could be made.

It was too soon to draw conclusions about the relation between light eflexion and skid resistance as Mr Harper had suggested. There was ttle comfort to be derived from the example which he had cited. It was ue that the smooth railway line reflected well, but the coefficient of fric-on between the smooth wheel and the smooth rail was only about 0·15 hen wet ; such a low value existant on a road would certainly lead to crop of skidding accidents.

Mr Carr had rightly stressed the importance of getting as light a coloured ggregate as possible, economics permitting. Surely there were many aces where the choice of a light-coloured aggregate instead of a dark one ould involve little or no additional expense. His suggestion for spraying hite material on to the surface as a temporary measure seemed well worth trial and it was intended to try it out.

However, it was clear that the effect of colour was confined mainly to ne parts of the surface which were close to a lantern. Most lanterns in ritain were of the high-angle beam type which were widely spaced and

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*** This contribution was submitted in writing upon the closure of the oral scussion.—Sec. I.C.E.

which threw little light on to the nearer portions of the surface. Thus, most lighting systems did not make the best use of a light-coloured surface. If the peaky surface dressing shown in *Fig. 9* had been as light in colour as the even concrete which was adjacent to it, it would still have appeared darker than the concrete, except in small regions near to the lanterns. Therefore, when a new lighting installation was being planned, the lanterns should be spaced as closely as economics would allow and should emit as little light at large angles to the downward vertical as would permit the bright patches to merge to a reasonable degree.

Dr Lee had asked whether it was more sound economically to suit the lighting to the surface or the surface to the lighting. There were so many factors involved that there was probably no general rule, but one should strive always to achieve a suitable combination. There were reasons for thinking that the surface was the more fundamental with regard to visibility. It could be shown theoretically that a reduction in road surface reflectivity had a greater effect on the visibility of pedestrians on the carriageway than a proportionate reduction in the power of the lamps. Even if the power of the lamps was raised to compensate for a reduction in surface reflectivity the lighting was more glaring and uncomfortable than previously. That point had been emphasized by Mr Carr. The case was somewhat analogous to interior lighting where it was not really satisfactory to paint the walls and ceiling dark and to instal extra lamps to supply extra light.

In the past the effect of the surface on visibility had rarely been appreciated ; usually the reflexion properties of the roads had received little or no consideration and the street lighting had been blamed for the poor results. Even when the effect of the surface had received due recognition, it had usually been assumed that it would be more costly to provide a better surface than better lighting. In fact, to install street lighting to last 15 to 20 years and to operate it for the same time might cost as much as to lay a rolled asphalt surfacing to last for the same period. Also, it might often be cheaper to lay a more suitable surface in the first place than to go on paying the extra running costs of additional or more powerful lamps.

The experimental surfacings at Slough were still too new to allow firm conclusions to be drawn, but the idea of coating with white asphalt the dressing of chippings to be applied to an ordinary rolled asphalt appeared to be particularly useful, because it afforded a method of improving, without great expense, the reflexion properties of new surfaces of that type. The reflexion properties were usually poorest when the surface was new.

Dr Lee and Mr Wood had emphasized the importance of making allowance for the wet condition on roads such as that shown in *Fig. 17*. On some streets, however, there were bright surroundings whose reflexions in the wet surface served to eliminate dark areas. In those exceptional conditions it might even be an advantage to have a low texture depth.

In stating that texture depth was the average thickness of the sand patch used in its measurement the Author had meant the average of the values obtained when the thickness of the sand was determined at a large number of points spaced equally over the patch. He had not meant, as Mr Harper suggested, one half of the depth of the valleys below the peaks. That explained why in the example which he used, the texture depth was two-thirds of the height of a pyramid and not one half. Both were measures of surface texture and it was interesting to note that somewhat similar differences existed between the measures of roughness of finish of metal surfaces used in different countries. But texture depth appeared particularly applicable to the study of wet road surfaces, because on a level impervious surface the texture depth corresponded to the rainfall required before the surface was flooded.

The Author had, in the past, been criticized for taking photographs from positions other than the normal driving position. But, as Mr Harper had pointed out, the view from the right-hand side of the road was also important.

He agreed with Mr Harper that the drawing given as *Fig. 4* in the advance proof was inadequate and had submitted another to take its place.

Mr Wood and Mr Lovell had suggested the use of the footpath and edges of the road as a background. The Author was sure that those were helpful in revealing pedestrians stepping on to the road and should receive more attention than they usually did. But he did not think that it would be wise to rely on them alone ; they formed much too small a part of the background of vertical objects on the road.

Mr Wood had reminded the meeting that it might be desirable to use one colour of light rather than another according to the backgrounds, other than the road surface, which it was desirable to light. The danger there was that that might involve too many changes of light.

There could be no doubt that the danger that Colonel Green cautioned against was a real one. But Group B lighting was designed on the assumption that drivers had to recognize that the onus was on them to use their headlights as and when required. The shorter lighting column of about 25 feet was employed to indicate that.

The closing date for Correspondence on the foregoing Paper has now passed without the receipt of any communication.—Sec. I.C.E.

CORRESPONDENCE

on two Papers published in Proceedings, Part II, June 1954

Road Paper No. 43

" The Design and Construction of a High-Speed Test Track for Motor Vehicles "†

by

Ralph Freeman, C.B.E., M.A., M.I.C.E., and John Antony Neill, B.Sc.(Eng.), A.M.I.C.E.

Correspondence

Mr G. E. Wild observed that, from comments made during the discussion on p. 205, it might have appeared to some that the compaction control exercised during the construction of the earthworks required an excessive number of staff and could, therefore, be criticised as an expensive luxury.

That view was not apparently shared by the Authors, even though the cost of the control work was of the order of 4 to 5 per cent of the value of the earthworks. It should be remembered that, in addition to the direct value of the soil tests in eliminating unsuitable borrow-pit materials and ensuring that the construction operations were completed satisfactorily, there was a less obvious advantage which could be inferred from the conclusion of the Authors " that the embankments can be relied on to support the rigid pavement without significant settlement." In achieving a satisfactory state of compaction, it was reasonable to assume that a high degree of uniformity in the fill had also been obtained. Both those factors would materially assist in minimizing damage to, and distortion of, the concrete paving. A saving in maintenance costs and a reduction in the time the track would be closed to traffic were, therefore, likely to be attained. Without control, the condition of the embankment would have been much more variable and the chances of occurrence of uneven settlement of the paving correspondingly greater. For high speed work the track would obviously have to be kept in first-class order, so that the

† Proc. Instn Civ. Engrs, Part II, vol. 3, p. 189 (June 1954).

expenditure on repairs during the years could amount to an appreciable sum. The interference with the operation of the track would cause further indirect losses of even greater consequence.

The Authors had given an outline of the functions of the control laboratory, which had been organized by Mr Wild's firm, in selecting fill material and checking the finished earthworks. He wished to give a more detailed account of the work that had been undertaken. All soil tests were made in accordance with B.S.1377, 1948.

A total of fifty-two samples was taken from the borrow-pits when they were opened up. For those samples, liquid and plastic limits, and the standard moisture density relations, were determined as well as the natural moisture contents. On the results of those tests a certain amount of borrow-pit material had been rejected as unsuitable and one borrow-pit area was, in consequence, not developed.

Routine checks on the fill material were carried out by taking the material after deposition on the bank, and the samples were subjected to the same treatment as those from the borrow-pits. A total of 143 such samples was taken during the building of the bank.

The compaction achieved in the banks at each stage of construction was checked by measuring the in-situ density after each layer had been compacted. Because the filling consisted primarily of clay, the simple core cutter method was employed and found to be entirely satisfactory. A total of 543 in-situ densities was measured during the initial construction of the banks and a further 209 determinations were carried out on the trimmed subgrade prior to sealing; in addition, about sixty in-situ densities were measured along the north-west straight and about fourteen in the first test section of the bank.

Number of staff employed on the control work varied according to the rate at which construction proceeded. When the laboratory was first opened, one senior assistant was able to deal with the sampling and testing, but during the months of July, August, and September when progress was rapid, two junior assistants were also sent to the site. For one fortnight in that period, a fourth operator was employed. After mid-October, however, only the senior assistant and one junior were retained on site, when they were concerned primarily with subgrade density tests. In addition, two labourers had been attached full-time to the laboratory during the busy period. Light transport was provided since it was impossible otherwise expeditiously to handle the number and weight of samples.

Generally, therefore, the control work was carried out by not more than one senior and two junior technical assistants, supplemented by two unskilled labourers. A typical day's work, once the embankment construction was proceeding normally, consisted of the examination of two or three samples of the borrow material taken either from the borrow-pit or from the embankment and tested for index properties

and moisture-density relationship and the measurement of about fourteen to twenty in-situ densities in the bank as it was compacted.

The Authors, in reply, thanked Mr Wild for his contribution; the figures which he had given were a valuable addition to the Paper.

Mr Wild had put forward the view that the amount of compaction control exercised should not be regarded as excessive, and had emphasized the importance of that aspect of the works. The Authors entirely agreed with that view, and would specify a similar intensity of control in future work of the same kind.

The cost of the tests represented only about 1 per cent of the total cost of the works. It might be argued that, once a proper compaction procedure had been established, more relaxation of the number of tests could have been allowed. At the most, a saving of about £1,000 might thereby have been achieved. Against that argument it should be remembered that the clay soil used for building the banks was not, and could not be expected to be, entirely uniform in character; and relaxation of testing might have resulted in significant changes being overlooked. The Authors were convinced that a saving of the order mentioned was not justified in comparison with the increased risk of heavy maintenance expenditure arising from uneven settlement, which such relaxation of testing would entail.

Railway Paper No. 52

"Railway Civil Engineering in the United States of America" †

by

Ian Macdonald Campbell, B.Sc.(Eng.),
and
Norman John Nicholls, B.Sc.(Eng.), A.MM.I.C.E.

Correspondence

Mr C. W. Clarke remarked that the Authors had emphasized that on American railways the permanent way constituted the greatest proportion of the assets, but that applied to any railway, and particularly to overseas railways. For example, the system with which Mr Clarke was connected (Western Australian Government Railways) had, in round numbers, 450 locomotives, 12,000 wagons and 400 coaches, and the total weight of steel involved was approximately 170,000 tons. The system had approximately 4,700 miles of track, which included 580 miles for

† Proc. Instn Civ. Engrs, Part II, vol. 3, p. 273 (June 1954).

yards and sidings, and the total weight of steel in the rails alone was about 450,000 tons. The latter figure excluded steelwork in bridges and other fixed structures.

Reference was made in the Paper to the use of long welded rails, but it should be made clear that in the United States the total track mileage exceeded 250,000 and normal rail renewals averaged about 10,000 to 12,500 miles per annum. The A.R.E.A. Report of 1953 showed that a total of only 247·5 miles of welded track had been laid in America, of which 208·6 miles had been laid in open track and 38·9 miles in tunnels. Welded track was first used in America in 1933 and the maximum mileage of welded track laid in any one year had been in 1950, when 50·25 miles had been laid. It was significant that the 155-lb. track being laid by the Pennsylvanian Railroad consisted of jointed track.

Perhaps Australia had laid a greater mileage of welded track than any other country. Welded track laid in Australian railways exceeded 1,500 miles, most of which had been laid during the past 8 years. Except for a few miles laid by the " Thermit " process, the welded joints were made by the flash-butt process. The ribbon lengths were generally 240 feet, the preparation of the ribbon lengths on each system being done at a central rail-welding plant, and the rails transported to site on trains consisting of 5 or 6 flat-top wagons, the rails being loaded in tiers three-high (see *Fig. 29*). Mr Clarke was not convinced that welded track was more economical for Australian conditions.

It was stated on p. 277 that joints were normalized by heating to about 1,500° C. Perhaps that should read " to about 1,500° F. (816° C.) ?

The switch-point guard (*Fig. 9*) served its purpose where the width of tires on all rolling stock was uniform, but on many overseas railways the tire width could vary up to 1 inch on different stock. In such cases switch-point protection could be given by the use of " Mack " point protectors or by arc-welding a short run of metal on the inner edges of the rails a little in front of the switch points.

Perhaps the most notable difference in the construction of points and crossings in British and American practice was that in British practice the inclination inwards of the stock rails, closure rails, and switch-blades was maintained throughout the crossing at 1 in 20. In American practice the stock rails, closure rails, and switch-blades were vertical, although the running rails ahead and in rear of the crossing were inclined inwards. That, though theoretically not so correct, led to a simpler design of crossing. The inclination run-out was taken up in the first few feet of the running rails adjacent to the crossing. The American system was adopted in Australia for the lay-out of points and crossings. In Australia the running rails were generally inclined inwards at 1 in 20, as in British practice. Although most American systems had now adopted a cant of 1 in 40, some of their lines still retained the vertical running rail. The only state to do so in Australia was Queensland.

Sleeper spacing and sleeper sections in Australia conformed more to British practice. During Mr Clarke's visit to Canada and the United States in 1953, the extensive use of 10-foot or 12-foot muskeg or " swamp ties " had been noticeable, especially in Canada.

Another striking feature was the heavy axle loads permitted on comparatively light track, especially in Canada. Whilst British practice was to confine locomotives of heavy axle loads to operation on heavy rail sections, in Canada a locomotive could operate on any section (subject to bridge loading restrictions) irrespective of axle load, but speed was restricted. It was not an uncommon sight to see a locomotive of 20-ton axle load operating on 60-lb. rails. The maximum speeds permitted by Canadian National Railways on different weights of rail and for various axle loadings were shown in *Fig. 30.*

In that respect Australian practice inclined towards American practice. For example, on the 3-foot-6-inch-gauge system of Western Australia, locomotives of 14-ton axle loading were confined to operation on 60-lb. or heavier track, but passing loops in station yards were still laid with 45-lb. rails, and such locomotives used the loops, the speed being restricted to 15 miles per hour.

Most of the existing railway bridges in Australia were of the ballast-deck timber trestle type, but whereas standard American practice was to lay the stringers direct on the caps, Australian practice was to retain the use of corbels. Within recent years the tendency had been towards concrete piers and steel-girder construction.

The Western Australian Government Railways had turned to Diesel traction extensively, and had on order from United Kingdom firms, 69 Diesel locomotives, some of which had already been delivered, the remainder being scheduled for delivery before the end of 1955, by which time Western Australia would have the largest fleet of Diesel locomotives in Australasia. The type of construction adopted for the Diesel locomotive depot at East Perth was shown in *Fig. 31.*

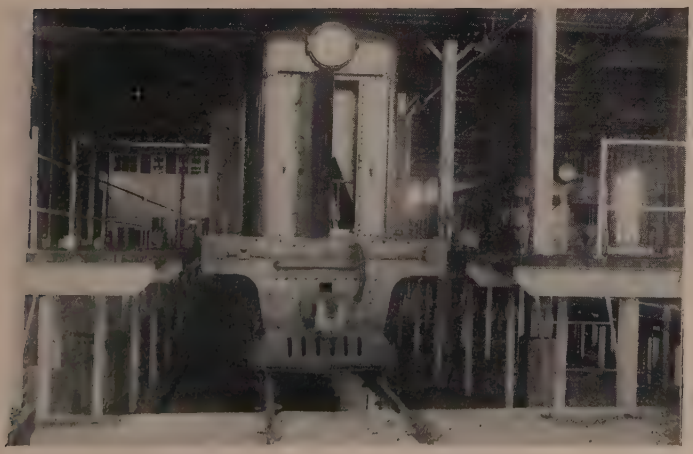
Mr Bernard Bramall observed that the question whether rail joints should be staggered or not had long been a contentious point of opinion between railway engineers on each side of the Atlantic and there was little point in reiterating the arguments in support of both practices. British practice had the advantage, however, that it had made possible the laying of prefabricated track, and it might be wondered whether spot replacing of sleepers in the United States would have continued as a general policy had prefabricated track been practicable.

Mr Bramall did not question the contention of the Authors that in some cases it had not been strictly economical to use prefabricated track, but the Authors would probably agree that it had economic value in a great many applications that should not be relinquished lightly. Often such re-laying in Britain provided an opportunity of renewing ballast to formation level; satisfactory sleeper spacing was simplified, and, as

Fig. 29

Train Load of Ribbon Rails being Dispatched from Rail Welding Depot

Fig. 31

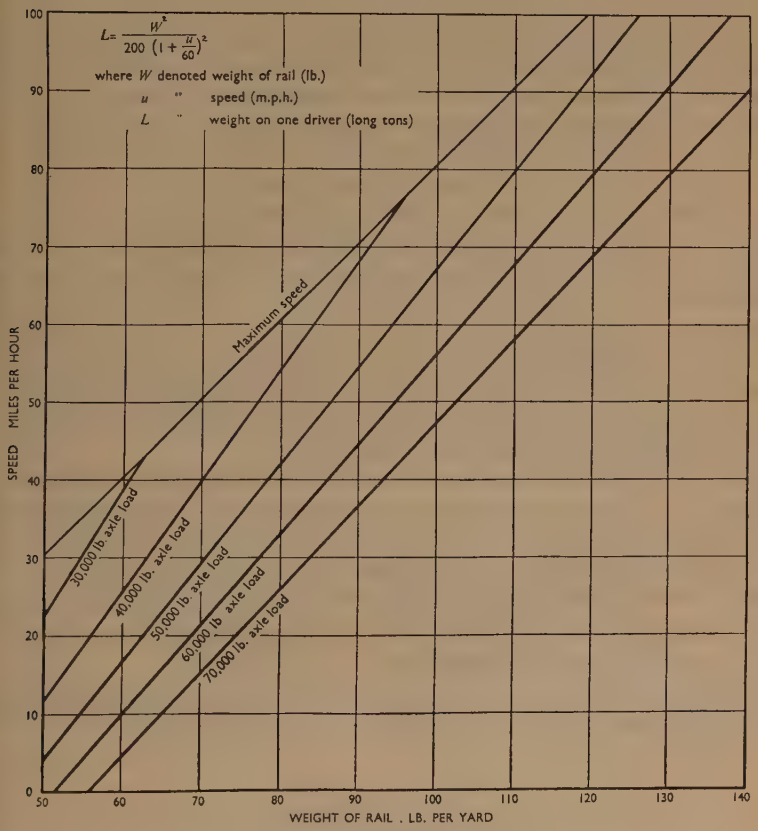


Diesel Locomotive Depot at East Perth, Western Australia.

vas mentioned in the Paper, sleeper spacing at the joints was given more onsideration in Britain.

General impact formulae for bridges required some caution. By ature they were statistical and, whilst they might be admirable under ertain conditions, it by no means followed that conditions in different

Fig. 30



Canadian National Railways

Calculated Maximum Sustained Permissive Speeds for Various Axle Loads and Rail Weights

countries were sufficiently similar. Such variables as prevailing axle oads, springing of rolling stock, locomotive balancing, and span/depth atio of bridges all had their effects. On that point reference might be made

to a Paper by Gribble,[3] which compared the B.S.S. formulae and the Government of India impact formulae, and showed the latter described the envelope of span/impact curves obtained on the Ceylon Government Railway fairly well. Much guidance might be obtained from the analytical results, which endeavoured to isolate the various factors, but any statistical approach would have to be in relation to British conditions.

A point which required serious attention was the suggestion that heavier beams, such as the American wide flange beams, would find useful application in Britain.

The Authors, in reply, observed that it was of course true, as Mr Clarke had remarked, that only a small percentage of welded track had been laid in America, but the technique had got past the stage of initial experiment, welded track having been laid in high-speed running lines. It would have been interesting to have learnt why Mr Clarke doubted the economy of welded rail for Australian conditions.

The Authors did not wish to condemn prefabricated re-laying entirely, since it had many useful applications, particularly in re-laying point and crossing lay-outs. If the type of renewal required (partial or complete) was first decided and the method second, there could be no objection to prefabricated re-laying when that was the most economical method of doing the work. The fault lay in a reversal of the order where the method was decided first. With prefabricated re-laying in fashion the type of renewal was, of necessity, complete renewal regardless of the physical condition of rail, sleepers, fastenings, ballast, etc.

The Authors agreed with Mr Bramall that the impact formulæ derived by the A.A.R. tests would not necessarily be suitable for British Railways' locomotives and bridges. In the opinion of the Committee for the Code of Practice for Simply Supported Steel Bridges, there was a need for research into the effects of modern locomotives on modern bridges. If such a programme was contemplated, the methods and apparatus used by the A.A.R. should be carefully considered.

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[3] Conrad Gribble, "Bridge Investigation, Ceylon Government Railways, 1949–1951." Proc. Instn Civ. Engrs, Part II, vol. 3, p. 106 (Feb. 1954).

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Proceedings, Part II, October 1953.

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